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"The Superstructure of the Island of Orleans Suspension Bridge, Quebec, Canada."

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TABLE OF CONTENTS.

Introduction.

The Island of Orleans, which is approximately 16 miles long and 5 miles in maximum width, is situated in the St. Lawrence river some 6 miles below the city of Quebec. It is a place of considerable historical and antiquarian interest, although not at present of any great economic significance. The population, which numbers some 4,000, forms an isolated community. The great majority of the people are French-Canadian and are devoted solely to agricultural pursuits, which have been their occupation since the advent of the first settlers 300 years ago. The island possesses an attractive topography and is increasingly popular as a summer resort.

Before the bridge described in this Paper was constructed, access to and from the Island of Orleans was effected in summer by a steam ferry plying between Quebec city and Ste. Petronille, I.O., and in winter by two or three ice-roads maintained across the North Channel and suitable only for horse-drawn sleighs. In spring and early winter communication was usually uncertain on account of masses of heavy ice, which, at these seasons, are kept in constant movement by the tidal water.

The construction of the Island of Orleans bridge was undertaken

* The meeting is recorded on p. 1 (June).—Sec. Inst. C.E.

by the Provincial Government of Quebec (through its Department of Public Works) in order to provide permanent and continuous communication with the island and to form a means of developing its real-estate and residential possibilities. It is interesting to note also that the project, undertaken as it was in a time of general distress, has largely been considered as an unemployment relief measure. For this reason the work has been spread over a period of more than four years, and local labour has been used to as great an extent as possible.

Fig. 1.

This bridge, which was opened to traffic on 6 July, 1935,[1] is built across the North Channel of the St. Lawrence, connecting the lower Quebec road at Montmorency Village with a point on the island road some three miles east of the village of Ste. Petronille (*Fig. 1*).

The North Channel, while neither as deep nor as wide as the main channel on the south side of the island, which is used by ocean shipping proceeding to and from Quebec and Montreal, is nevertheless capable of considerable development as a shipping route.

At low tide this channel has a width of 2,000 feet with a maximum depth of 60 feet (Fig. 2, Plate 1) of almost fresh water at the site of

[1] The painting and cable-wrapping was not completed until the following month.

the bridge, and is flanked on either side by mud flats of large extent. The tidal range is 20 feet and there is a reversing current of some 4 knots. The Federal Government requirements for clearance under the bridge call for an unobstructed fairway of at least 600 feet in width, with a minimum headroom of 106 feet at high water.

The location is very exposed and is subject to almost continual winds blowing either up or down the river at all seasons of the year. In winter the entire river north of the island is frozen over, the ice over the deep channel being carried up and down vertically with the tide, and the shallower flanks of the waterway being completely frozen to the bottom.

Beyond the narrow coastal flats on each side of the North Channel, the land rises steeply; on the island it forms a plateau about 250 feet above the river, and on the mainland it leads to a terrace of similar height, behind which lie the foothills of the Laurentian plateau. The Montmorency Falls, where the river of that name flows into the St. Lawrence from the north, are within a mile of the bridge. The site of the bridge is visible for many miles both up and down the river and is of considerable beauty.

The complete bridge has a total length of about $2\frac{3}{4}$ miles, and consists of:

(*a*) The central portion crossing the main channel; this is a suspension bridge with two flanking spans, built on six piers which carry the anchorages, the cable-bent posts and the main towers. The length from centre to centre of the anchorages is 2,370 feet (Fig. 2, Plate 1).

(*b*) Two series of 150-foot steel Warren-truss deck-spans on concrete piers, leading up to each end of the suspension bridge. These viaducts are 1,350 feet long on the island approach and 900 feet long on the Quebec side.

(*c*) Two concrete viaducts consisting of 60-foot beam-spans. These lead on to the steel viaducts, and are each 600 feet long inclusive of the abutments.

(*d*) About $\frac{1}{2}$ mile of rock-faced earth embankment at each end of the bridge. At Montmorency Village the embanked roadway joins the lower Quebec road; on the Island of Orleans the embankment connects with an approach roadway. A toll-house is situated near the west end of the bridge.

(*e*) The new approach-road built to connect the bridge with the main belt-road of the island, which at this point runs along the top of a fairly sharp bluff some 100 feet above the level of the river. This new roadway is over $\frac{1}{2}$ mile in length.

This Paper deals with the central portion of the bridge, the principal dimensions of which are shown in Fig. 2, Plate 1. The approaches, although of considerable length, are in general of conventional type. and do not possess any outstanding features either of design or of construction. The bridge roadway is 20 feet wide, with two 5-foot sidewalks.

The suspension type of structure was adopted, after considerable study entailing the investigation of cantilever, arch, and simple-span designs, as being the most suitable from the view-points of economy, adaptability to foundation conditions, aesthetic fitness and facility of erection.

It may be noted here that solid limestone shale was available for foundations on the Island side of the channel. On the north side, the rock was entirely out of reach, and foundations had to be made on deep sand; the sand, however, was of a satisfactory nature, being of coarse grain and firmly packed.

The main proportions of the bridge, which, it may be noted, is the first long-span suspension bridge to be entirely designed, fabricated and erected in the Dominion of Canada, are in general accord with modern practice. Table I shows comparative figures of the Island of Orleans bridge and several other suspension structures of a similar size. With the exception of the cables of the Mount Hope and Portsmouth bridges, which are built of parallel wires, the cables of all the bridges shown in the Table are of parallel-strand construction, the strands being shop-manufactured and pre-stressed. The strands of the Cologne-Mülheim bridge, which is of the "self-anchored" type, are of "locked-wire" construction.

It should be noted that, in accordance with Canadian practice, weights are expressed throughout this Paper in "short tons" of 2,000 lbs. avoirdupois.

Design and Fabrication.

Loadings and Specifications.

The suspension bridge was designed to suit the following loadings, based on reasonable weights and spacing of vehicles. As is usual in the case of this type and size of structure, they were drawn up by the engineers especially to meet the individual requirements of the project.

The following alternatives were used for live load:

(*a*) A uniform "normal" load (N) of 900 lbs. per linear foot of bridge, or a uniform "congested" load (C) of 1,200 lbs. per linear foot of bridge. These loads were "discontinuous" and so distributed as to produce the

theoretically maximum effects at any point along the span. No impact effect was considered in conjunction with these loads.

(*b*) A local load consisting of 80 lbs. per square foot on the sidewalks together with two conventional 15-ton trucks (with 30 per cent. impact-allowance for wheel loads) abreast on the roadway. This loading was used in the design of the floor-slab and supporting steelwork, the web-members of the stiffening-trusses, and the suspenders.

For wind effects, a lateral load (W) of 400 lbs. per linear foot of bridge (composed of 300 lbs. per linear foot on the trusses

TABLE I.

Bridge.	Date of completion.	Centre span: feet.	Side-span if suspended: feet, inches.	Centre-span sag-ratio.	Width centre to centre of cables: feet, inches.	Truss depth: feet, inches.
Portsmouth, Ohio, U.S.A.	1927	700	350 0	1/10	31 6	14 0
Waldo - Hancock, Bucksport, Maine, U.S.A.	1931	800	350 0	1/10	28 0	9 0
Grand'Mère, Que., Canada . . .	1929	949	—	1/10	26 0	12 2
Cologne - Mülheim, Germany . . .	1929	1,033	—	1/9·1	72 10	19 8 (girder)
Isle of Orleans, Que., Canada . . .	1935	1,059	417 6	1/9·5	31 8	13 0
Maysville, Ky., U.S.A.	1931	1,060	465 0	1/10	28 0	14 0
Mount Hope, R.I., U.S.A.	1929	1,200	504 0	1/10	34 0	18 0
St. John's, Portland, Oregon, U.S.A. .	1931	1,207	430 0	1/10	52 0	18 0
1st Narrows, Vancouver, B.C., Canada . . .	projected	1,500	575 0	1/10	40 0	18 0

and 100 lbs. per linear foot on vehicles) was adopted. In the central span 270 lbs. per linear foot[1] was assumed to be resisted by the truss lateral system, and 130 lbs. per linear foot carried by the cables. In the side span, the whole pressure was taken by the truss lateral system. The direct wind load on the main towers was taken as ($1\frac{1}{2} \times 30$) lbs. per square foot of vertical projection.

[1] This figure was deemed reasonably approximate for the case in point. It was obtained from the "restitution" formula given by Dr. D. B. Steinman, "Suspension Bridges," p. 133. (New York, 1929.)

Temperature effects (T) due to variations of — 80° F. and + 60° F. from a normal temperature of + 60° F. were considered. In the particular case of the design of expansion-joints, the lowest temperature was taken as — 40° F.

In regard to general detail and workmanship, the Canadian Engineering Standards Association's Standard Specification for Steel Highway Bridges (A6—1929) was used, except as otherwise ordered by the engineers. Rivets throughout, except for certain small details, are $\frac{7}{8}$ inch in diameter.

Floor-Slab.

A departure from the conventional floor-design was made on this bridge by the introduction of a patent "Teegrid" slab of composite steel and concrete construction, in which welded connections are used throughout. The total weight of this slab per square foot of roadway is 50 lbs., whereas an equivalent slab of the ordinary reinforced-concrete type would weigh up to 80 lbs. per square foot, as it is about $6\frac{1}{2}$ inches thick. The consequent saving in weight, reflected as it is in a reduction of weight in the towers, cables and trusses, and in a relief of the anchorage-pull at the pier-bases, was the dominating reason for the choice of this form of decking.

The "Teegrid" slab (*Fig. 3*) is manufactured of structural-steel "T"-sections 3 inches by 3 inches and 1·38 square inches in area, placed side by side with their flanges welded together at intervals. Half-round cross-bars, $\frac{1}{2}$ inch in diameter, running at right angles to the tees and spaced at 4-inch centres, are welded into slots in the stems of the tees. The spaces between the tees are filled with concrete, the surface being made level with the tops of the stems of the tees and cross-bars, which thus act as an armour against abrasion from traffic. Two incidental advantages of the "Teegrid" floor are that no formwork is required for the pouring of the concrete, and that the steel grid alone is capable of supporting without damage any temporary traffic and erection plant.

The "Teegrid" sections are 4 feet wide and 20 feet $2\frac{3}{4}$ inches long. They lie athwart the bridge, welded to the tops of the stringers and to each other, and were cambered 2 inches in the shops to conform to the profile of the roadway. The longitudinal continuity of the slab is broken at intervals of 32 feet (every eighth grid section being unattached to its neighbour), in order to provide for relative movement of parts of the slab due to changes in shape of the suspended trusses.

In computing the strength of the slab, the moment of inertia and the position of the neutral axis were obtained by the use of standard

formulas for calculating the properties of a reinforced-concrete beam. For the slab used, the moment of inertia per foot of width is 95 inch^4

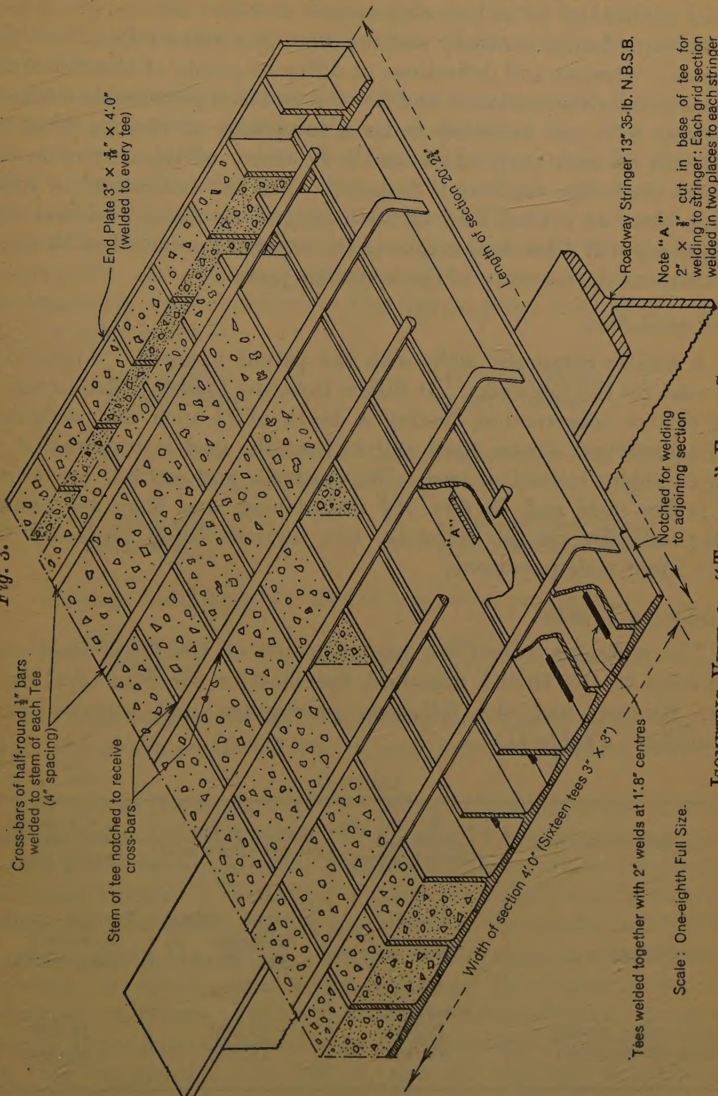

Fig. 3.

ISOMETRIC VIEW OF "TEEGRID" FLOOR-SLAB.

units, the neutral axis being situated 0·96 inch from the backs of the tees.

Before accepting this type of flooring, the engineers arranged for a full-size test to be made. A section of "Teegrid" 4 feet wide and

12 feet long was mounted on stringers at 5-foot centres, and equal loads were applied from a hydraulic jack through two sections of solid rubber tire 15 inches wide spaced at 6-foot centres, one 5-foot span being loaded centrally and the other at a point 1 foot from the centre. Stresses and deflections at different points of the slab were measured for various loads, and it was found that permissible working stresses were not exceeded under a total load of 15 tons (that is, 7·5 tons on each tire), which load is in excess of the maximum for which the bridge roadway is designed. The first indications of yield took place at a total load of 30 tons, and no failure, beyond the appearance of a few hair-cracks in the concrete, occurred at 35 tons, which was the capacity of the hydraulic jack.

Sidewalk-Slab.

A similar composite slab, with the patent name of "Anglgrid," is used for the sidewalks. It differs from the roadway slab in being of lighter construction, having a depth of 1½ inch and weighing 27 pounds per square foot, and in that the grid is constructed of small angles instead of tees. The sidewalk grid-sections are 2 feet 6 inches wide and 15 feet 11¾ inches long, and they are placed longitudinally on the sidewalk, two sections being necessary for the 5-foot wide pavement.

Stiffening-Trusses.

The stiffening-trusses are manufactured of structural steel conforming to the British Standard Specification for Structural Steel for Bridges, and the following permissible unit stresses were specified. (Table II.)

TABLE II.

Loading combination.	D. or W.: lbs. per square inch (Normal values).	D.W. or D.N.T.: lbs. per square inch.	D.N.T.W. or D.C.T.: lbs. per square inch.
Tension	18,000	Normal values increased in ratio 25/18.	Normal values increased in ratio 28/18.
Compression . . .	$\left(17{,}000-60\frac{l}{r}\right)^*$ with 15,200 as maximum.		
Shear (shop rivets) .	13,500	17,000	17,000
Shear (field rivets) .	12,000	15,000	15,000

The main cables and stiffening-trusses were designed in accordance with the "deflection" theory of stress-analysis, which takes into account the comparatively large distortion of a suspension structure

* l/r represents the ratio of the length of the member to its radius of gyration.

under varying conditions. This theory was advanced in the first case by Mr. J. Melan in 1888, and has since been developed for practical use by various bridge engineers, notably by Mr. L. S. Moisseiff, whose particular application of the theory was followed in this case.

Preliminary rough values for cable-area, truss-depth, weight and moment of inertia, obtained by use of the much shorter "approximate" or "elastic" method of analysis, were used for the first trial computations by the "deflection" method, and were altered in a series of progressive approximations. The final values of these quantities, which, when used in conjunction with the foregoing permissible unit stresses, yielded results in close accord with the assumptions, were as follows. The figures given refer to one truss and/or cable only :—

Area of cable	41·3 square inches.
Depth of truss	13 feet 0 inches.

Average moment of inertia of the truss, taken over the middle 80 per cent. of the truss-length :

Centre span	4,200 inch²-feet² units.*
Side span	2,200 inch²-feet² units.*

Average dead loads per foot of horizontal projection :

Centre span	1,645 lbs.
Side spans	1,437 ,,

The dead loads of the actual structure were found to differ slightly from the above estimated values, and were made up as shown in Table III.

The curves drawn in *Fig. 4* (p. 367) give the maximum values of bending moments and shears, for various loading conditions, at all points across the central span. Bending moments indicated as positive are those from such conditions of temperature and loading as cause maximum compression in the top chord-section considered. As an instance, using the N.T.W.† loading combination, maximum compression in upper chord-section CU17–CU19 obtains under the following conditions. Both side spans are unloaded and a "normal" load is applied to the central span from panel-point 0 to panel-point 18 : the temperature is + 120° F., and full wind-load is applied to

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* The above figures for the moments of inertia of the trusses were obtained by adding 8 per cent. to the moment of inertia for chords only, this being a conventional allowance for the stiffening effect of the web system and connection details.

† These symbols are explained on pp. 360 and 361.

the central span in such a direction that the truss in question is to windward. Shears indicated as positive are those which produce compression in the end post L_0U_1.

Each one of these curves was drawn by calculating the maximum value of the bending moment or shear separately for each of a series of panel-points across the span. Calculations of maxima for as many as ten points on the more critical curves were made.

The computation of the maximum value of the bending moment or shear at a given panel-point is in itself somewhat laborious. The value of "H," the horizontal component of the cable-tension for the load and temperature combination under consideration, has

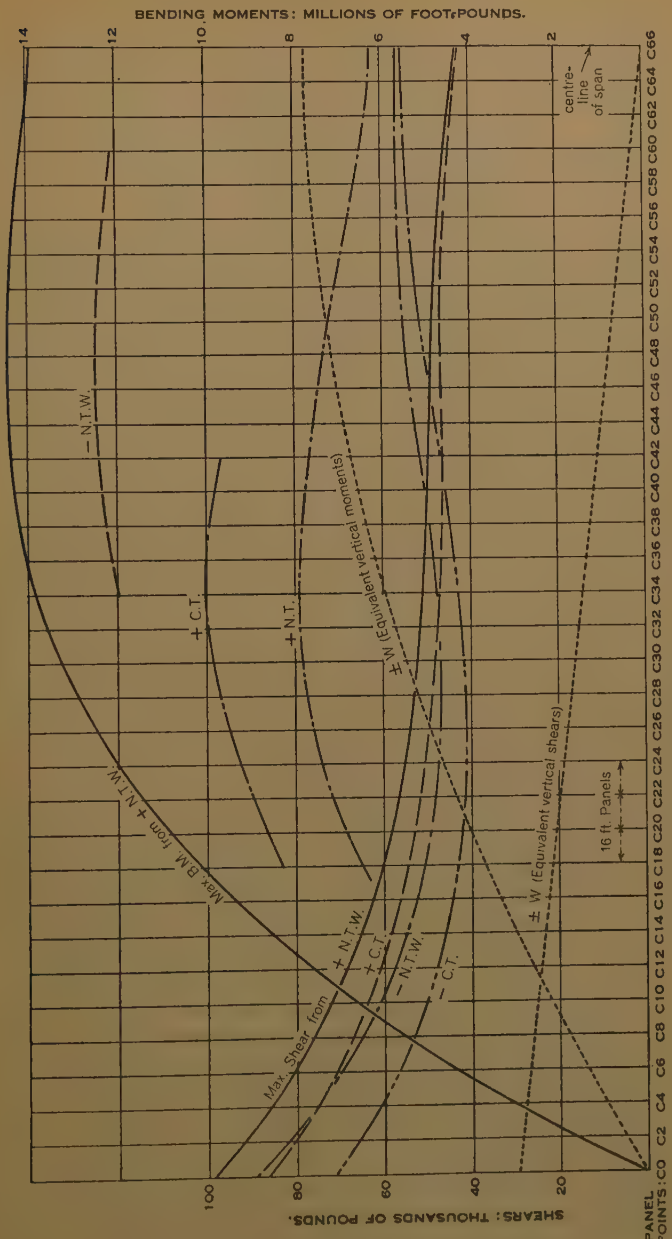
TABLE III.

	Central span.	Side span.
"Teegrid" and "Anglgrid" floor, steel	265 lbs.	265 lbs.
"Teegrid" and "Anglgrid" floor, concrete	384 ,,	384 ,,
Floor steel	262 ,,	262 ,,
Trusses and laterals	525 ,,	343 ,,
Suspenders and sockets	10 ,,	15 ,,
Cable-bands	12 ,,	12 ,,
Cable	150 ,,	155 ,,
Wrapping	14 ,,	15 ,,
Fence, conduits, lighting	24 ,,	24 ,,
Paint	8 ,,	8 ,,
Total weight per foot of horizontal projection.	1,654 ,,	1,483 ,,

first to be obtained by a series of trials; from this the value of the bending moment or shear is obtained by the use of the appropriate constants of integration. Except in the simpler cases (for example, maximum central bending moments) several different "loaded lengths" have to be investigated in order to determine that which yields the maximum effect. A small hand-operated calculating machine, capable of multiplying nine significant figures, was used to facilitate computation, and was found to be invaluable.

Chord loads due to lateral forces were computed on the assumption that the upper chords of the stiffening-trusses participate equally with the lower chords as chords of the lateral wind-trusses, in the following manner. Primarily, the lower chords of the stiffening-trusses, being in the same plane as the lateral web system, receive the lateral chord loads. The stiffness of the loaded cable, however, prevents any relief which would take place were the trusses free to bend up or down in their own vertical planes: the upper chords

Fig. 4.



MAXIMUM BENDING MOMENTS AND SHEARS IN CENTRAL SPAN.

are thus constrained to share the lateral chord loads, and shearing forces due to the wind are also set up in the web members of the truss.

On this assumption, the equivalent vertical load for wind on either truss was

$$\frac{270 \times 13}{2 \times 31{\cdot}67} \text{ lbs. per linear foot,}$$

and the bending moments (shown in *Fig. 4* as "equivalent vertical moments") appear as quantities, varying parabolically along the span, which may be combined directly with the vertical bending moments expressed in the other curves. In a similar manner, the shears due to the lateral wind have been expressed as "equivalent vertical shears." Similar curves were drawn for the side span, this case being less complicated in that the curves of maximum bending moments are all parabolic.

A typical section of a stiffening-truss is shown in Figs. 5, Plate 1. The trusses are of the Warren type with verticals, the chords being of box section with the legs of the angles turned outwards in the upper chord and inwards in the lower chord. The maximum gross sectional area of the upper chord is 53·85 square inches for the central span and 31·88 square inches for the side span, while that of the lower chord is 45·82 square inches for the central span and 28·60 square inches for the side span. The upper chords are 1 foot 2½ inches deep from back to back of the angles, and are stiffened with latticing and batten-plates. The lower chords, 1 foot 1½ inches deep over the angles, are stiffened with batten-plates only.

The chord-splices occur every 32 feet, those of the upper chord being staggered with those of the lower, and were all accurately machined and shop-assembled in order to ream the rivet-holes in the splice-material. Chord face-to-face lengths were such as to provide the correct shape of the finished truss for conditions of normal temperature and dead load only. The diagonal web-members, with the exception of the end posts, which are of double channel-section, each consist of two angles tied together at intervals with batten-plates. The hanger- and post-members are of "H" section built up of a plate and four angles, and all are truly vertical under normal conditions notwithstanding the varying grade of the truss. The truss panels are 8 feet in length, the hangers being spaced at 32-foot centres.

Under conditions of dead load only, with normal temperature, the weight of the trusses is carried entirely by the cables, and there are no end reactions. Such positive or negative reactions as occur under other conditions are transferred to the towers by means of

rocker-posts pin-connected to the lower chords as shown in Figs. 5, Plate 1. These rocker-posts maintain the level of the roadway at the ends of the trusses (the trusses being discontinuous at the towers), and at the same time permit of longitudinal truss-movements due to changes in temperature and loading.

The lateral wind systems are of "X" bracing situated under the deck, and the members consist in general of two angles with their shorter legs in contact. Where necessary, they are supported from the stringers by light hangers made from $\frac{3}{4}$-inch rod. The lateral gusset-plates are riveted to the undersides of the lower chords and floor-beams. The lateral wind-thrust at each end of the trusses of the central span is transferred to the towers by means of a "nose" connection which slides in a slot provided on the wind lateral strut of the tower, a sliding connection being necessary on account of the longitudinal movements of the truss. The position of the sliding nose in relation to the tower strut is indicated in the drawing of the tower (Figs. 15, Plate 1). The side-span lateral nose is pin-connected to the tower-strut, expansion taking place at the cable-bent connection, where a sliding fitting is provided.

Floor-System.

The floor-slab, previously described, is carried on a system of floor-beams and stringers. The floor-beams are at 16-foot centres, being situated at each vertical member of the truss. The floor-beams (Figs. 6, Plate 1) are plate-web girders 3 feet in depth. They are normal to the grade of the roadway and consequently they make a varying angle (corresponding with the grade at the point in question, which is always small) with the vertical members into which they are framed. The roadway-stringers, of 16 feet span, are British Standard Beam Sections 13 inches deep, and are spaced at 5-foot centres. They rest on top of the floor-beams and are so raised on shims that their top flanges conform to the shape of the cambered floor-grids.

The longitudinal sidewalk-grids are carried on transverse supports, which are "T" sections made by cutting standard 10-inch beams longitudinally. These are spaced at 8-foot centres, the intermediate supports being carried on light sidewalk-stringers, and are inclined from the horizontal to give a drainage slope to the sidewalk. The roadway proper is bounded by steel curbs of angle section cut from standard 15-inch channels. The toe of the curb-angle is flush with the sidewalk surface, and there is a space between the heel of the angle and the surface of the roadway to provide straight drainage without the use of gutters, grids, or downpipes. The height of the curb is $6\frac{1}{2}$ inches.

Fences.

Light fences of wire mesh are provided, weighing about 15 pounds per foot of length. They are of No. 6 S.W.G. galvanized wire, fabricated into a 2-inch diamond mesh; the fences are supported at top and bottom by sections of $1\frac{1}{2}$-inch diameter galvanized pipe, welded in the field to small lugs shop-riveted to the web-members of the truss.

Main Cables.

The stress in the cable depends primarily upon the length and sag of the cable in the central span, the side-span sags being so arranged that the horizontal component H of the cable-stress is constant throughout the entire length of the cable under conditions of dead load only with normal temperature; the towers are then vertical, there being no horizontal reaction to them from the cables. In the design calculations it is further assumed that H remains constant throughout the cable for any other condition of loading, the towers being flexible and offering but slight resistance to horizontal forces at the saddles.

The maximum value of H occurs for the case of "congested" loading over the entire bridge together with the condition of minimum temperature, and was computed by the deflection theory to be:

$$H_{\text{D.C.T.}}{}^{*} = 2{,}773{,}000 \text{ lbs. per cable.}$$

The greatest cable-stress occurs at the point of maximum inclination of the cable to the horizontal, which is at the end of the side-span bight where the cable enters the tower saddle. This slope being about 26 degrees, the maximum cable stress is:

$$P_{\text{D.C.T.}} = 3{,}087{,}000 \text{ lbs. per cable.}$$

The specified working stress for the cable was 75,000 pounds per square inch, and the necessary cable area was consequently 41·17 square inches.

Each main cable consists of thirty-seven twisted-wire strands of $1\frac{3}{8}$ inch diameter, laid parallel to one another to form a hexagonal section. This section is filled out to circular shape with cedar-wood fillers impregnated with linseed oil, and is then bound with No. 9 S.W.G. soft annealed double-galvanized wrapping-wire. The finished diameter is 10 inches. (Fig. 7, Plate 1.)

The steel for all cable-wire, and also for all suspender-rope wire, was specified by the engineers to be "acid open-hearth cold-drawn bridge-wire manufactured in accordance with the best current

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* The suffix D relates to dead load, and N, C, W and T relate to the loadings and conditions explained on pp. 360 and 361.

practice," and to be "the product of works of established reputation for the kind and character of wire specified." The following is a synopsis of the requirements of the specification in regard to the properties of the wire.

The steel was required to show, on ladle-test, not more than the following percentages of elements:

Carbon	0·85
Phosphorus	0·04
Sulphur	0·04

Specimens of the finished galvanized wire were required to have a minimum average tensile strength of 220,000 pounds per square inch of gross section (including galvanizing), with a minimum of 215,000 pounds per square inch.

It was specified that the yield-point, defined as being at an elongation of 0·75 per cent., should occur at a stress of not less than 165,000 pounds per square inch of gross section.

The minimum permissible elongation at failure was 4 per cent., with a reduction in area of at least 30 per cent.

The galvanizing of the wire was required to withstand from two-and-a-half to four (depending on the diameter of the specimen) 1-minute immersions in a defined standard solution of copper sulphate.

The wire was to be capable of being coiled cold around a mandrel of one-and-a-half times its own diameter without showing signs of fracture, and around a mandrel of five times its own diameter without developing cracks in the galvanizing visible to the naked eye.

No torsional requirements were specified.

In regard to the cable-strands, the engineers' specifications required a minimum ultimate strength of 217,000 lbs. per strand and a minimum yield-point (defined as the total stress at which the elongation of the specimen between gauge points 100 inches apart was 0·7 inches) of 167,000 lbs. The modulus of elasticity was required to have a minimum value of 24,000,000 lbs. per square inch after pre-stressing. The requirements of the specification relating to pre-stressing, together with the results of that operation, are set out in the Appendix.

The type of strand construction approved and employed is depicted in Fig. 8, Plate 1*. The strand consists of thirty-seven main wires and six smaller filler-wires, the diameters being as shown in the figure. The lay of the eighteen outer wires is opposite in direction to that of the interior of the strand, which consists of the remaining nineteen

* See p. 443 and *Fig. 42.*—Sec. Inst. C.E.

wires and six fillers. This type of construction was used in order to produce a strand as free as possible from any tendency to twist when subjected to a change of load, and the method was very successful. It is of interest to note that each complete strand, including as it does two separate lengths and directions of lay, was manufactured from the wire on one machine and in one operation; each wire is of the full length of the strand, there being no splices whatever in the main cables.

In the building-up of the cable itself, the layers of strands were arranged to be alternately of left and right lay as regards their outer wires, in order to produce a more satisfactory bedding of the cable strands at the saddles, where the vertical bearing-pressures of strand upon strand are the greatest. The contact between each strand and those immediately over or under it is hence of the nature of "line" rather than "point" bearing.

The breaking strength of the wire used for the cable-strands was obtained from specimens taken from both ends of every coil of wire supplied by the manufacturers. About six thousand four hundred tests were thus made. Ultimate stresses varied from 215,000 to 249,000 lbs. per square inch, the great majority lying between 220,000 and 235,000 lbs. per square inch. The average load at the specified yield-point was about 175,000 lbs. per square inch, and the elongation averaged about 6 per cent.

Twenty-four test specimens were cut from the strands after pre-stressing, each strand having been manufactured a few feet longer than the final length required. The ultimate strength varied from 238,700 to 269,000 lbs. per strand, the average value being 258,500 lbs. The average load at the specified yield-point was 170,100 lbs. The total gross sectional area of a strand (including galvanizing) was measured and was found to be about 1·19 square inch. The strand efficiency, obtained by a comparison of the ultimate breaking load of the strand with the sum of the breaking loads of the individual wires (all obtained by test), was computed to be 96 per cent.

The length of cable between the anchorages under normal conditions was calculated from the actual dead loads of the structure, obtained from the detailed drawings. The cable was considered as an elastic rope hung from the points of contact with the saddles, and the length, slope, tension and elongation under dead load of each separate chord-length between suspender-points was easily determined. The lengthening due to local sag between the suspender-points was neglected. The back-stay length was computed as for a parabolic curve. The strands were all made of equal length, the very small discrepancies introduced by this assumption being taken

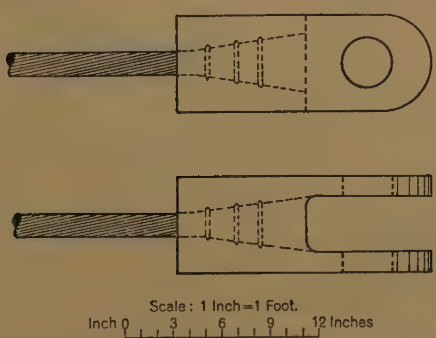
up in the individual adjustments of the anchorage bolts. The length of the central strand under dead load at normal temperature is 2,468·392 feet from face to face of the sockets.

Strand Socketing.

The cable-strand sockets, shown in *Figs. 9*, are steel forgings 6 inches square and 1 foot $3\frac{3}{4}$ inches in overall length, each provided with two lugs $1\frac{1}{4}$ inch thick for assembly, by a 3-inch-diameter pin, to the anchorage eyebolt. The conical interior cavity, or "basket," of the socket, into which the end of the strand is secured, is 8 inches long, with a diameter increasing from $1\frac{1}{2}$ inch to $3\frac{3}{8}$ inches. Three annular grooves $\frac{1}{4}$ inch deep are cut in this conical inside surface, their purpose being to increase the bond between the socket and the soft-metal mould.

The procedure in socketing a strand was as follows. The socket

Figs. 9.



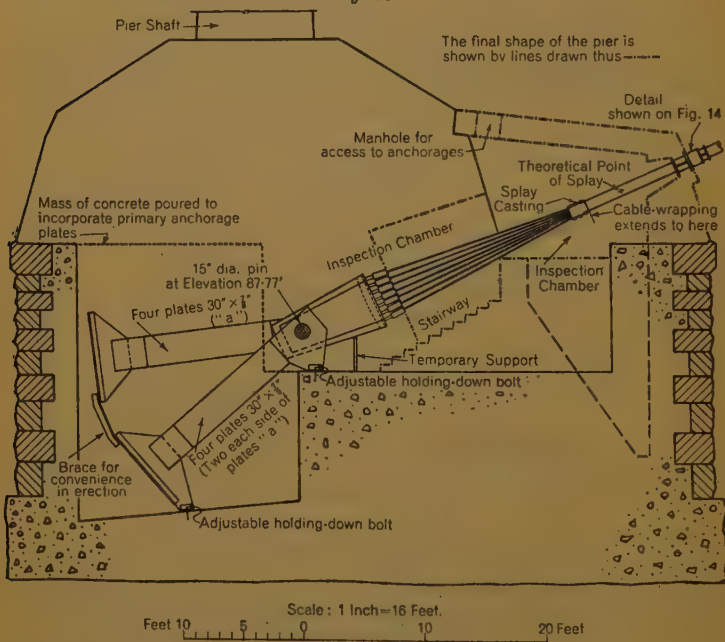
having been slid back along the strand, the latter was seized with No. 18 S.W.G. soft steel wire in order to preserve the lay of the wires. After cutting (which was done with a power hacksaw, as the wires were very hard) the wires of the strand were released for a length of 8 inches, "broomed" out, and cleaned in benzine. No acid was used.

For holding the broomed end in place in the socket, an apparatus was devised whereby the end of the strand could, while being centralized with respect to the socket-basket, be adjusted to its required position with great accuracy by means of a rack and pinion arrangement. This appliance held the socket rigidly in a vertical position, and was erected on a platform high enough to enable the standing part of the strand to be led up to it vertically, thus ensuring that the socket was square to the axis of the strand. The socket, thus held with its lugs upward, and having been previously tinned on its inner surface, was heated to a temperature of 250° F. The

basket (containing the broomed end of the strand, correctly adjusted in regard to final length) was then filled up with one continuous pour of 99·75-per-cent.-pure zinc at a temperature of about 850° F. During the pouring of the metal, the socket was vibrated by hand-hammering, which was continued until the mass ceased to be molten.

The zinc was heated by a stove located on the socketing-platform and its temperature was tested before each pour by means of a thermo-electric pyrometer.

Fig. 10.



SECTION AA OF *Figs. 11.*

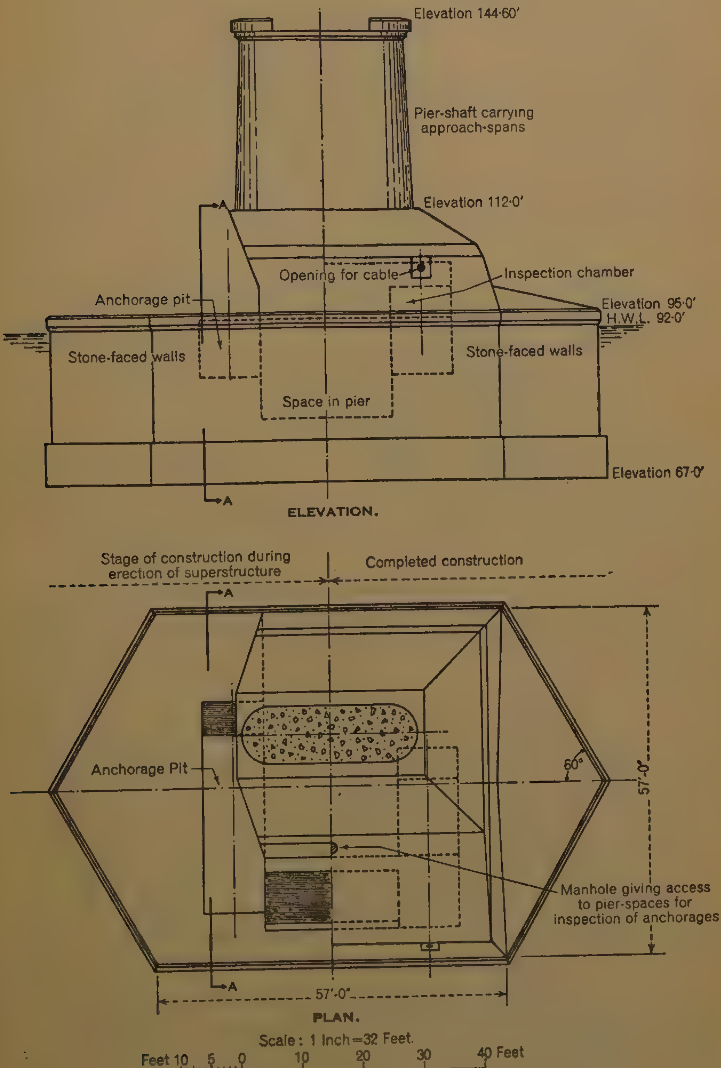
Anchorages.

Following the usual practice for a bridge of this type, the strands of each cable, in order to provide room for socket-connections, are splayed out at a convenient point near the anchorage. The strand-sockets are secured individually to a series of fan-shaped steel plates, which in their turn are connected through a main anchorage-pin to other plates solidly anchored into an adequate mass of concrete. A notable new detail, however, has been incorporated into the anchorages of this structure, at the point of connection of the strand-sockets with the plates.

The essential innovation consists of a heavy steel slab, machined on the side facing towards the anchorage-pin to a spherical surface

generated from a centre point at the theoretical point of splay of the cable-strands. Forged 3-inch eye-bolts of special high-tensile

Figs. 11.



steel, pin-connected to the sockets of the strands, pass through this slab normal to the spherical surface. The bolts are secured by standard 3-inch hexagonal nuts, each of which bears on a small planed

area machined on the spherical surface at right angles to the direction of the bolt. The slab for each anchorage was machined and drilled as a whole, and was then sliced into six parallel sections for assembly between the seven fan-shaped anchor-plates. A general view of the anchorage assembly is shown in *Figs. 10* and *11.* Reference to the construction of the anchor-piers is made later.

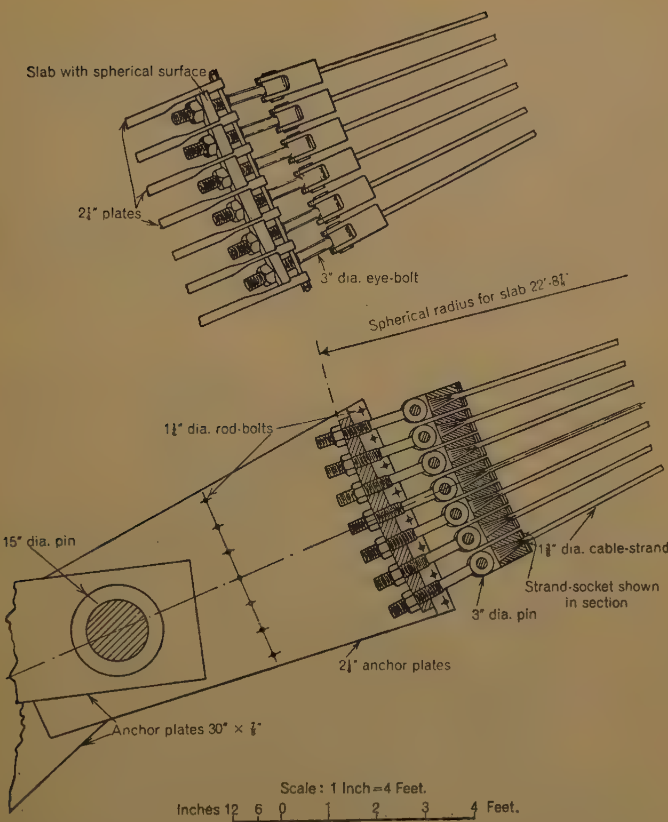
At each anchorage there are eight primary plates, 30 inches by $\frac{7}{8}$ inch, connected with grillages which engage with the mass concrete of the pier. These are arranged in two groups converging to the main pin, which is 15 inches in diameter. There are seven secondary fan-shaped plates, $2\frac{1}{4}$ inches thick, which alternate with the primary plates at the pin. All the plates are bored to $15\frac{1}{32}$ inch diameter. The secondary plates are notched near their outer edges to receive the six sections of shaped slab, their thickness being reduced to 1 inch by the notching. They are prevented from spreading sideways by fifteen $1\frac{1}{4}$-inch rod-bolts, the nuts of which are secured by "Palnut" patent lock-nuts. The 3-inch eyebolts passing through the shaped slabs are secured between the jaws of the strand-sockets by 3-inch-diameter pins, which are held by cotter-pins made from $\frac{3}{8}$-inch rod. The detail of this assembly is given in *Figs. 12.* The anchorages were designed to withstand a maximum cable tension of 3,042,000 lbs. (computed for the D.C.T. combination of loading).

The cable strands are caused to converge to the normal cable shape by a splay casting situated about 24 feet from the anchorage pin. The inside faces of this casting are shaped in smooth curves to lead the strands together without sharp changes in alignment. The two halves of the casting are squeezed together on to the strands by six $1\frac{1}{2}$-inch bolts. As there is an odd number of strands in the cable, arranged in a manner which it is not practicable to reproduce at the anchorage, the various strands are splayed out as shown diagrammatically in *Figs. 13.* It will be noted that there is consequently one unused section of the bearing-slab.

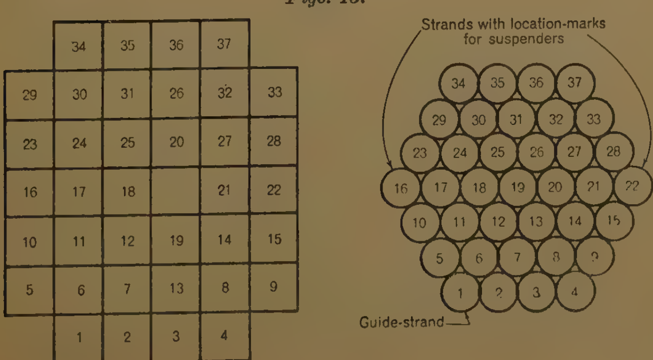
The compacted cables pass out from the pier-cavity through steel "eyelet" castings set into the concrete breast-wall. The eyelet-castings have an inside diameter of 11 inches in order to leave the cable free for small vertical movements caused by changes in temperature and loading conditions. Flexible copper flashings are employed to prevent moisture from entering the pier-cavity through the space between the cable and the eyelet casting. Details of the eyelet castings and flashings are shown in *Fig. 14* (p. 378).

Access to the interior of the pier for inspection of the anchorages is provided by a manhole, located as shown in *Fig. 10.*

Figs. 12.



Figs. 13.

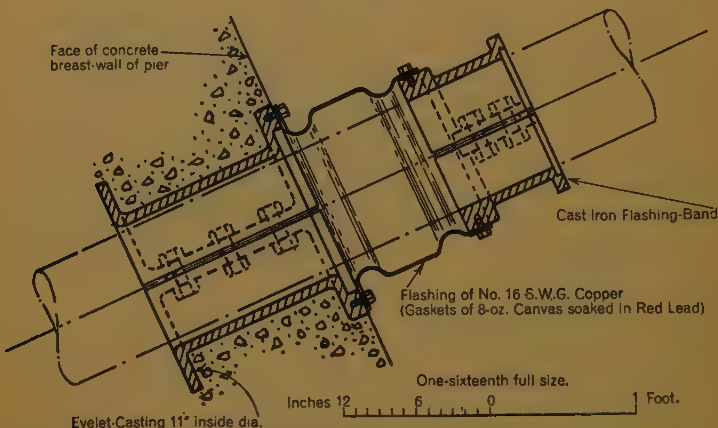


Main Towers.

The appearance of a suspension bridge is inherently graceful on account of the shape of the curves of the cables and the grades of the roadway-platform. Nevertheless, the form of the towers, and particularly of the tower-bracing, dominating as it does any view of the bridge except a square side-view, has no small effect upon the æsthetic quality of the structure.

The towers of the Island of Orleans bridge, each consisting essentially of a pair of flexible steel columns rigidly anchored to the pier and connected by adequate cross bracing to resist side forces, are shown in Figs. 15, Plate 1. Both columns and bracing were de-

Fig. 14.



signed and detailed with special regard to their appearance as the main supporting members of the bridge and for the important part they play as outstanding features of the landscape. The " X " form of sway bracing, employed in the towers of many suspension bridges, is that which expresses to the eye in the most straightforward manner its function as a web system for resisting lateral forces. This type of bracing is used here, but the strict austerity of the design was softened by an arching of the top strut and of the diagonal members immediately above the roadway. The tower-columns are shaped in accordance with structural requirements except for a widening of the base in order to accentuate the appearance of stability at this point. The height of the towers from the centre-line of the cables to the bearing-surface of the piers is 220 feet 6 inches, and the distance between the centres of each pair of columns varies from 31 feet 8 inches at the top to 42 feet at the bottom.

The material used in the main column-members and lattice-bars

is structural silicon steel made in accordance with the specification given in Appendix VI of the Canadian Engineering Standards Association's Standard Specification for Highway Bridges (A6–1929), which requires a tensile strength of from 80,000 to 95,000 pounds per square inch and a minimum yield-point of 45,000 pounds per square inch. Messrs. Dorman, Long's "Chromador" steel was largely used for this work. Ordinary structural carbon steel is used for the tower-bracing and other details.

To determine the permissible compressive stress in the silicon-steel, the following formula [1] was used:

$$f_c = 22{,}000 - \frac{320 \times l_1 \times l_2}{l \times r} \text{ lbs. per square inch,}$$

with a maximum value of 19,000, in which:

l_1 and l_2 denote the distances from the top and bottom of the column respectively to the section under consideration.

$l = l_1 + l_2$, the length of the column.

r denotes the radius of gyration at the section considered.

This specified unit stress was used for load-combination D.N.T. For combinations D.N.W.T. and D.C.T., the permissible unit stress was increased by 15 per cent.

The design of the columns was governed throughout by stresses obtaining under the D.N.T. combination of loading, as will be seen from the design-figures in Table IV. In this Table, the values shown for direct vertical load were obtained from the condition of loading (*a*), in which all spans are covered with the uniform load (either N. or C.) at the lowest possible temperature.

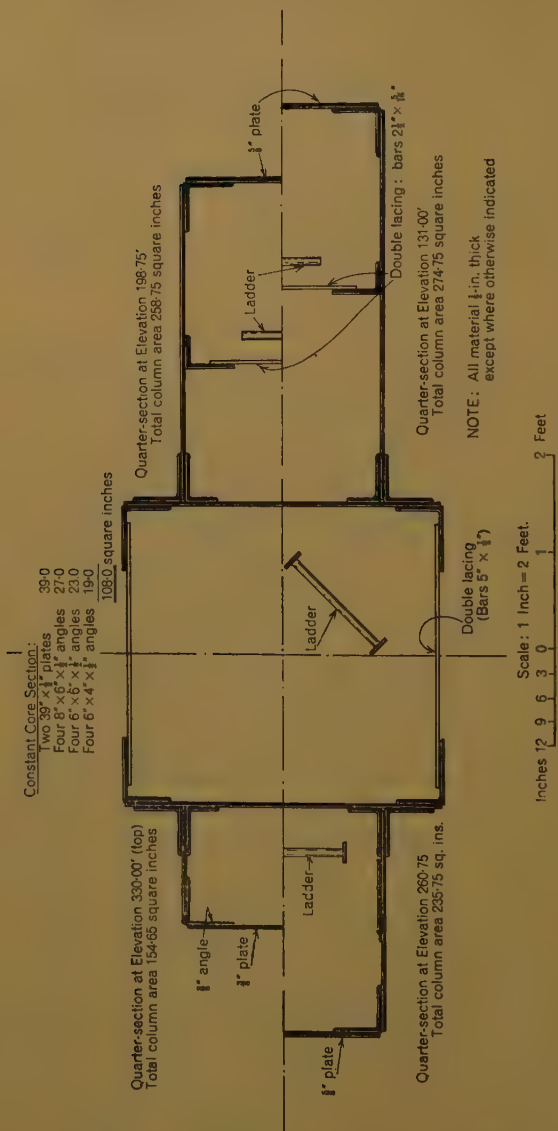
The bending moments shown were computed for case (*b*) in which the side span adjacent to the tower in question is unloaded, the other two spans being fully loaded (either N. or C.) at the highest temperature. For case (*b*) with N.T. loading, the top of the tower is deflected 1·00 foot away from the shore, and the unbalanced horizontal pull in the cables at the tower saddle is 8,980 lbs. With C.T. loading, the tower deflection is 1·25 foot and the unbalanced pull is 10,200 lbs. The bending moments are due to the consequent eccentricity of the direct loads, and to the overturning effect of the unbalanced pulls in the cables. The direct loads of case (*a*) were used in calculating the bending moments, although the theoretical direct loads of case (*b*) are slightly smaller.

Wind-loads in the columns and in the cross bracing of the towers were computed graphically in the ordinary way, the shear in any

[1] This formula was developed by Mr. P. L. Pratley, M. Inst. C.E., and first used in this design.

panel being assumed to be taken half by one diagonal, in tension, and half by the other, in compression. The local stresses in the

Fig. 16.



Cross Sections of Tower-Column.

fibres of the columns, due to bending as a continuous beam under lateral wind-loading, were found to be negligible, as also were the stresses from longitudinal wind.

Table IV.

Elevation : feet.	Properties of section provided.					D only.	Deflection = 1·000 foot. Pull = 8,980 lbs. Case of D + N + T.					D + N + T + W.			Excess over compressive stress, f_c: per cent.	Deflection = 1·250 foot. Pull = 10,200 lbs. D + C + T.					Excess over normal permissible compressive stress, f_c: per cent.
	Area : square inches.	Moment of inertia, I : inches⁴.	Section, modulus S : inches³.	Radius of gyration, r : inches.	Permissible str ss, f_c: lbs. per square inch.	Load : kips.	Load : kips.	Direct stress, f_d : lbs. per square inch.	Moment : kips-feet.	Bending stress, f_b : lbs. per square inch.	Total stress, f : lbs. per square inch.	Load : kips.	Direct stress, f_d : lbs. per square inch.	Total stress, f : lbs. per square inch.		Load : kips.	Direct stress, f_d : lbs. per square inch.	Moment : kips-feet.	Bending stress, f_b : lbs. per square inch.	Total stress, f : lbs. per square inch.	
114·00	340·76	1,253,306	12,420	60·6	19,000	2,160	2,678	7,850	4,385	4,240	12,090	3,790	11,110	15,350	—	2,827	8,290	5,463	5,280	13.570	—
131·00	274·75	499,764	7,375	42·6	19,000	2,132	2,650	9,640	4,221	6,890	16,530	3,455	12,560	19,450	2·3	2,799	10,160	5,260	8,560	18,720	—
146·10	270·75	465,955	7,080	41·4	19,000	2,110	2,628	9,680	4,047	6,840	16,540	3,433	12,660	19,500	2·6	2,777	10,210	5,061	8,590	18,800	—
161·20	266·75	433,179	6,790	40·2	18,460	2,088	2,606	9,760	3,869	6,840	16,600	3,411	12,760	19,600	6·1	2,755	10,290	4,849	8,570	18,860	2·0
176·30	262·75	401,980	6,500	39·1	17,620	2,067	2,585	9,820	3,651	6,700	16,520	3,390	12,870	19,570	11·0	2,734	10,380	4,584	8,450	18,830	6·9
191·40	258·75	372,379	6,225	37·9	16,950	2,046	2,522	9,750	3,408	6,600	16,350	2,940	11,360	17,960	6·0	2,667	10,270	4,283	8,250	18,540	9·2
198·75	256·75	358,736	6,100	37·3	16,660	2,012	2,512	9,780	3,276	6,400	16,180	2,930	11,410	17,810	7·0	2,657	10,330	4,123	8,100	18,430	10·6
219·75	248·75	305,395	5,570	35·0	16,020	1,984	2,484	10,000	2,884	6,200	16,200	2,804	11,270	17,470	8·9	2,629	10,550	3,636	7,820	18,370	14·5
240·75	241·75	262,855	5,165	33·0	15,820	1,957	2,457	10,170	2,441	5,690	15,860	2,777	11,490	17,180	8·4	2,602	10,790	3,087	7,160	17,950	13·3
261·75	233·75	224,654	4,800	31·0	16,090	1,931	2,431	10,410	1,949	4,890	15,300	2,621	11,720	16,610	3·2	2,576	11,000	2,472	6,180	17,180	6·8
277·50	211·75	193,578	4,410	30·2	16,770	1,913	2,413	11,400	1,553	4,250	15,650	2,603	12,300	16,550	—	2,558	12,090	1,972	5,370	17,460	4·0
298·50	172·60	128,331	3,240	27·3	17,970	1,892	2,392	13,860	984	3,630	17,490	2,472	14,310	17,940	—	2,537	14,690	1,251	4,640	19,330	7·5
314·25	150·60	98,792	2,690	25·6	19,000	1,878	2,378	15,760	537	2,390	18,150	2,458	16,310	18,700	—	2,523	16,750	682	3,040	19,790	4·0
330·00	144·60	84,480	2,510	24·2	19,000	1,866	2,366	16,390	72	340	16,730	2,366	16,390	16,730	—	2,511	17,360	92	440	17,800	—
332·50	—	—	—	—	—	1,861	2,361	—	0	—	—	—	—	—	—	2,506	—	0	—	—	—

The cross section of the column at various levels is shown in *Fig. 16*, and will be seen to be composed of three rectangular box-sections of plates and angles. The centre box is of constant size throughout (3 feet $3\frac{1}{2}$ inches by 3 feet $0\frac{1}{2}$ inch from back to back of the angles) and of constant sectional area. The two side boxes are of constant width (2 feet from back to back of the angles), and vary in depth regularly from 1 foot $2\frac{1}{2}$ inches at the top to 4 feet $0\frac{1}{2}$ inch at elevation 131·00, below which level their depth increases rapidly until, at the top of the pedestal (elevation 114·00) the total column width is 16 feet 10 inches.

The tower-columns were each fabricated in six sections, with field splices at elevations 161·58, 190·21, 222·29, 258·17, and 295·75 respectively. The cross section at each splice was machined as a whole after the piece in question was fully riveted, and the abutting surfaces of each splice were brought together in the fabricating-shops, the rivet-holes of the various splicing-pieces being reamed while the pieces were in position. This work was done with great care, the machining being so accurate that the largest feeler that could be inserted between abutting surfaces was of $\frac{6}{1000}$ inch thickness.

Inspection-ladders are provided inside the box-sections, the one in the core-section extending the full length of the tower and giving access both to the pier and to the cable-saddle. The ladders are reached by a door in the column at roadway-level. A handrail is provided on the top strut of the tower-bracing to facilitate access to the saddles.

Tower-Shoes.

The four tower-shoes or pedestals are built up entirely of steel slabs welded together, and represent a notable departure from the traditional type of cast-steel pedestal. The base of the pedestal is a horizontal $1\frac{1}{2}$-inch plate, 18 feet long and 8 feet 2 inches in maximum width. Vertical webs ($1\frac{1}{2}$ inch thick underneath the main core-section of the column, and $1\frac{1}{4}$ inch thick under the side sections) carry a top bearing plate, $1\frac{1}{4}$ inch thick, which is inclined to the horizontal in order to be normal to the batter of the column. The plates are welded together throughout with $\frac{1}{2}$-inch fillet-welds, suitable openings being left to permit access for interior welding. The depth between the top and bottom surfaces of the pedestal after planing varies from 2 feet on the outer side to 1 foot 10 inches on the side towards the tower centre-line, and the pedestal shape in plan conforms to the outline of the cross section of the base of the column.

The completed pedestal (*Fig. 17*) was subjected to a normalizing

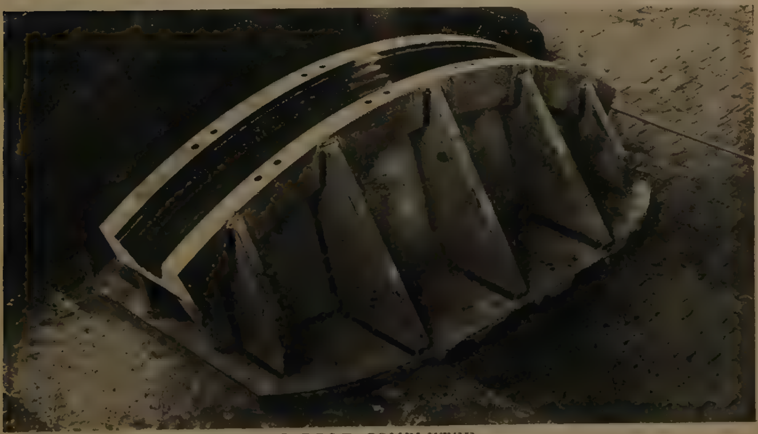
Fig. 17.

ALL-WELDED TOWER-SHOE.

Fig. 18

ALL-WELDED TOWER-SADDLE DURING CONSTRUCTION.

Fig 19.



SADDLE COMPLETED.

Fig. 25.

ERECTION OF MAIN TOWER ON PIER No. 21.

process to relieve the internal stresses set up by the heavy welding. It was maintained at a temperature of 1,150° F. in an oil-fired furnace for a period of from 6 to 7 hours. Each pedestal rests on a double layer of heavily-painted $10\frac{1}{2}$-ounce canvas, and is anchored down to the pier by twenty-eight anchor-bolts of $1\frac{1}{2}$ inch diameter, the tower-column being riveted to the top plate of the pedestal. The pedestals weigh 9 tons each.

Tower-Saddles.

The tower-saddles are also of all-welded construction instead of being steel castings, as is the more usual practice. The construction of a saddle is shown in *Figs. 18* and *19.* The principal part of the saddle is a steel forging 16 inches wide, $6\frac{1}{4}$ inches deep, and 7 feet 4 inches long, shaped to the curve assumed by the cable at the tower. The top surface of this forging is at the elevation of the centre-line of the cable, and is curved to a radius of 6 feet $0\frac{5}{8}$ inch. The forging is machined out to the profile of the cable, and is grooved to fit the individual outer strands. The base plate of the saddle is $1\frac{3}{4}$ inch thick, 4 feet $4\frac{1}{2}$ inches wide, and 6 feet $4\frac{1}{2}$ inches long. The two longitudinal supporting webs are $1\frac{1}{2}$ inch thick, and the five transverse ribs are of 2-inch and $1\frac{1}{2}$-inch material. The main welds connecting the saddle-forging to the supporting webs are $\frac{5}{8}$-inch reinforced fillet-welds, making a joint of 100 per cent. efficiency.

The base and webs were first welded together, and this part of the assembly was stress-relieved by the process described in connection with the main shoes. The forging was then welded into position, and the completed saddle again subjected to the normalizing process. The upper half of the cable, which is clear of the actual saddle, is held in place by eight keeper-castings, also grooved to fit on to the strands. The total weight of each main saddle is 3 tons. It is secured to the top of the tower-column by twenty-two $1\frac{1}{4}$-inch-diameter turned bolts.

Finials.

The tower-columns are each finished off with a cap or finial (shown in Figs. 15, Plate 1) built up of No. 10 U.S.G. copper-bearing steel plate, and provided with a side door and a sliding cover to permit of the inspection of the cable and saddle. Flashings are provided at the points of entry of the cable.

Besides providing a weatherproof cover for the cable at this point, the finials have also an architectural function. They are designed with a view to obviating the otherwise anomalous appearance of the junction of the comparatively slender lines of the cable with the

much heavier profile of the column, and also in order to give a definite appearance of finality to the tops of the towers without at the same time producing the effect of being merely ornamental.

Cable-Bents.

Owing to the flat topography of the immediate site of the suspension bridge and to the consequent location of the anchorages at a comparatively low level, supports had to be provided at the ends of the suspended side spans in order to give the necessary elevation to the cables at these points. (Fig. 2, Plate 1.) Each cable-bent (*Figs. 20*) consists of two vertical posts, pin-connected to anchorages on the cable-bent piers and carrying cable-saddles bolted to their tops, The posts, placed 31 feet 8 inches centre to centre, are connected by sway-bracing and by a double cross girder. The post-sections are 41 feet 6 inches long.

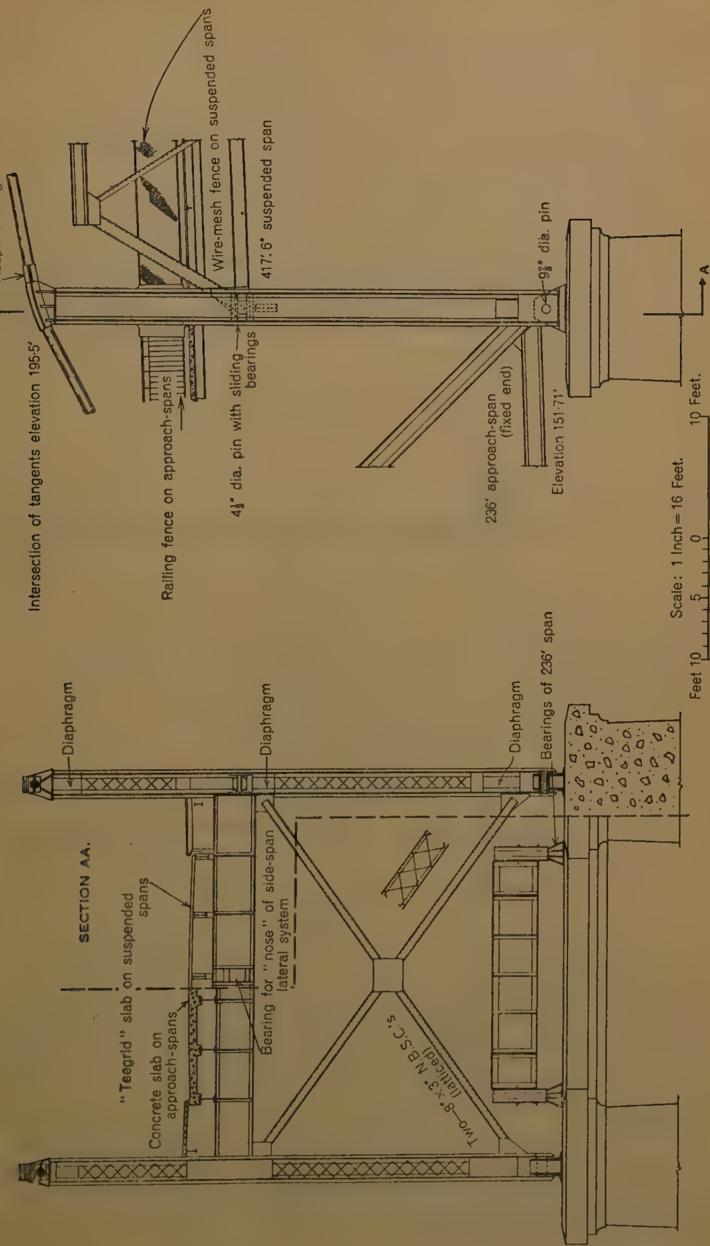
The trusses of the 236-foot approach deck-spans are at 20-foot centres, their fixed ends resting on shoes on the centre-line of the cable-bent pier. These trusses have no vertical end-posts, and the end-stringers are carried on the cable-bent cross-girder, as are also the end-stringers of the suspended side spans. All stringers at this point have sliding bearings to allow for movements of the cable-bent and of the end of the side span. The side-span trusses are pin-connected to manganese-bronze plates which can slide between parallel horizontal steel plates on the insides of the post-sections. The lateral system of the side span is provided with a " nose " which slides in a hole provided in the cable-bent cross girder. Floor-movements are provided for as described later.

The cable-bent posts each carry a vertical load from the cable, due to the change in slope of the latter at the saddle. The maximum value of this load, occurring under the D.C.T. combination of loading, was calculated as 910,000 lbs. (including the dead load of the cable-bent itself). The cross section of the post is a plate-and-angle box-section with diaphragms, as shown in *Fig. 21* (p. 386). The total cross-sectional area is 66 square inches, the required area being 63 square inches. The cable-bents are fabricated of structural carbon steel, the permissible working stress used for compression being $(17{,}000 - 60l/r)$ lbs. per square inch.

Cable-Bent Saddles.

The saddles at the cable-bents, although smaller, are similar in construction to those at the main towers (described on page 383). The cable is carried on a steel slab, grooved out to the profile of the

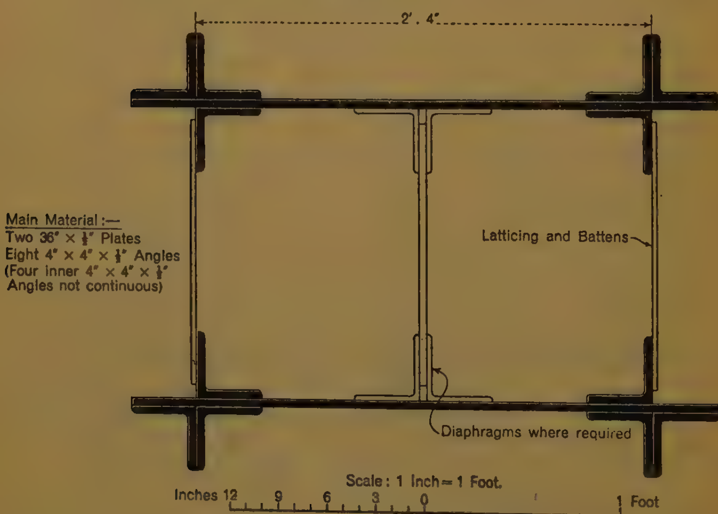
Figs. 20.


Keeper-casting
A
Intersection of tangents elevation 195·5'
Railing fence on approach-spans
Wire-mesh fence on suspended spans
4⅛" dia. pin with sliding bearings
417'.6" suspended span
9⅞" dia. pin
236' approach-span (fixed end)
Elevation 151·71'
Scale: 1 Inch = 16 Feet.
Feet 10 5 0 10 Feet.
SECTION AA.
Diaphragm
Diaphragm
Diaphragm
Bearings of 236' span
"Teegrid" slab on suspended spans
Concrete slab on approach-spans
Bearing for "nose" of side-span lateral system
Two—8"×3" N.B.S.C.'s (latticed)

cable and curved to a radius of 13 feet $5\frac{13}{16}$ inches on the centre-line of the cable. This slab is welded to three transverse ribs $1\frac{1}{2}$ inch thick, and two longitudinal webs $1\frac{1}{4}$ inch thick, these supporting members being in turn welded to a base-plate 3 feet 1 inch by 2 feet 5 inches by $1\frac{1}{4}$ inch, which is bolted to the top of the cable-bent post. The saddles were heat-treated after fabrication, in order to relieve internal stresses due to welding.

Unlike the tower-saddles however, these saddles are provided with a heavy cap-piece formed of another steel slab, 16 inches by $5\frac{9}{16}$ inches, grooved to the profile of the top half of the cable and shaped to fit the cable-curve ; these caps are secured to the saddles proper by $1\frac{1}{4}$-inch-diameter bolts of high-tensile Mayari steel. Each saddle with cap weighs 1·5 ton.

Fig. 21.



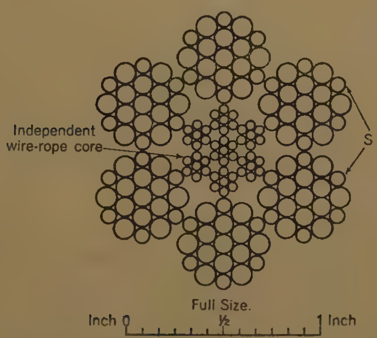
The cable-bent post is maintained in its perpendicular position solely by friction between the cable and the saddle The maximum unbalanced tension at the saddle, due to change in the slope of the cable, is about 250,000 lbs., and the friction-grip, in addition to that due to the cable-reaction, is aided by the eight bolts holding down the solid saddle-cap. As a further safeguard, it was later decided to assemble a casting, similar to a suspender-rope casting, on the cable in contact with the cable-bent saddle. The position of this casting is indicated in *Figs. 20.*

Suspender-Ropes.

The following combinations of loading were considered in the design of the suspender-ropes.

		lbs.
Average dead load on one suspender in central span arising from 32 feet of bridge is		47,000
Live load on one suspender from the maximum condition of uniform loading (combination C.T.) is $\frac{8fH}{l^2} \times 32$ (where H_{CT} denotes 714,000 pounds and f denotes the cable sag) is . . .		18,300
Alternative live load from hypothetical case of two double panels of discontinuous truss, loaded with 80 pounds per square foot on sidewalks and two conventional 15-ton trucks abreast. For one suspender, this gives :		
Sidewalk	12,800	
Trucks $24{,}000 + 6{,}000 \times \frac{22}{32}$	28,100	
Impact (30 per cent. of truck load)	8,400	
Total		49,300
The maximum combination of dead and live loads was thus .		96,300

Fig. 22.



The engineers' specification called for "$1\frac{3}{8}$-inch-diameter galvanized suspender-ropes . . . of six strands of nineteen wires each around an independent wire-rope centre of seven strands of seven wires each. . . . The minimum ultimate strength . . . shall be 148,000 pounds when tested around a sheave with a radius of 7 inches." It was further specified that the suspender-rope should be manufactured in long lengths and pre-stressed : the pre-stressing requirements and procedure are dealt with in the Appendix.

The suspender-rope submitted and approved was of Warrington construction, made up of six strands surrounding an independent wire-rope centre (*Fig. 22*). The wire-rope centre is built up of seven strands each composed of a core-wire of 0·064 inch diameter,

surrounded by six outer wires of 0·056 inch diameter. The six rope-strands (S) are each made up of a core-wire of 0·105 inch diameter, six inner wires of 0·098 inch diameter, and twelve outer wires having diameters of 0·104 inch and 0·077 inch alternately. Individual wires were spliced when necessary by brazing during the spinning of the rope. The rope itself and the independent wire-rope centre are both of right-handed regular lay, in accordance with common practice. The requirements in regard to manufacture, strength and galvanizing of the wire were substantially the same as those for the cable-strand wire. Tests were made on each end of each manufactured length of wire to determine the breaking strength; this varied from 220,000 to 250,000 lbs. per square inch, and averaged about 235,000 lbs. per square inch.

The suspender-rope, having a total length of 11,547 feet, was manufactured in one continuous operation and was cut into five long lengths for pre-stressing. On this account it was deemed necessary to test only five specimens of rope, four of them being cut from the ends and two intermediate points of the manufactured lengths after these had been pre-stressed, while the fifth was taken from a preliminary sample of rope prepared by the contractor.

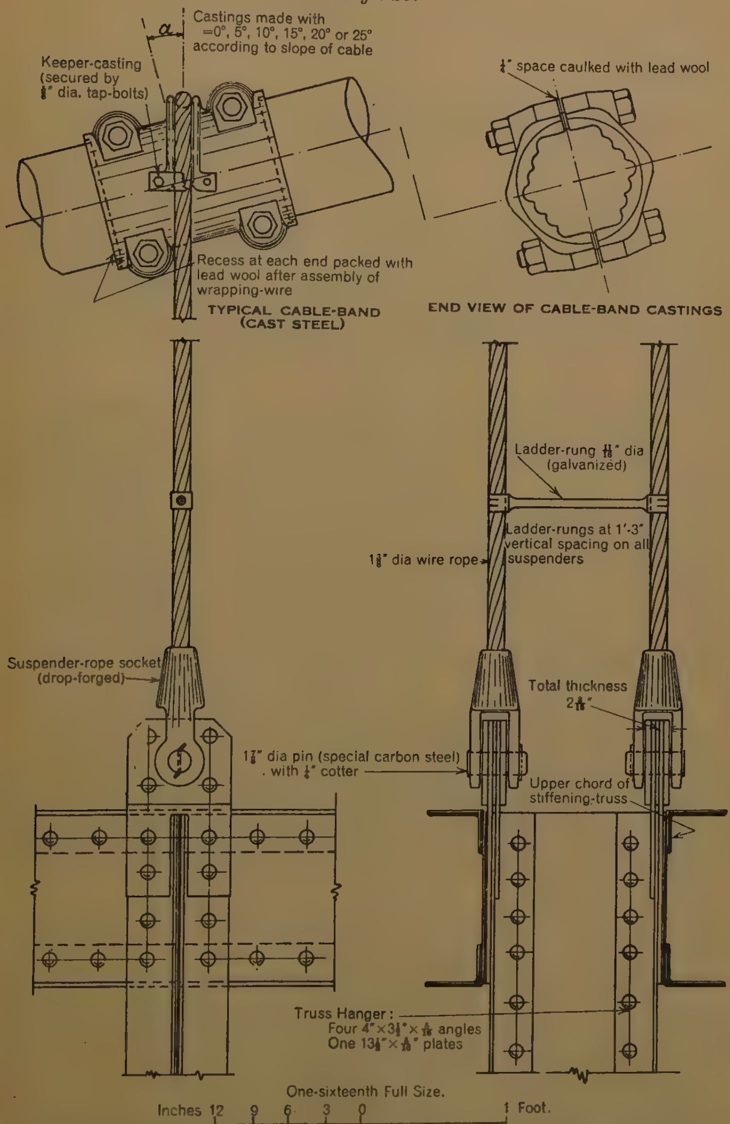
One of these test-lengths, each of which was of the specified length of 100 inches between the faces of the sockets, was tested in direct tension and failed at 198,400 lbs. The average total area of the wires in the rope being 0·926 square inch, this indicated an efficiency of $\frac{198{,}400}{0{\cdot}926 \times 235{,}000}$, or 91 per cent.

The other four lengths were tested over a specified sheave having a tread diameter of 14 inches. Two were tested to destruction in the only available testing machine of sufficient strength (at the Ontario Department of Mines, Toronto) and failed at 324,800 lbs. and 325,600 lbs. respectively. The specified ultimate strength was 296,000 lbs. In view of the uniformity of these two breaking loads, the engineers permitted the contractor to make the remaining two tests on a local testing-machine of 300,000 lbs. capacity. Neither of the specimens failed at this load. The final lengths of the suspender-ropes, from centre to centre of the socket pin-holes, range from 8·93 feet to 236·37 feet under dead load. *Figs. 23* show the assembly of a suspender-rope to the cable and to the stiffening-truss. It will be noted that there is no provision for any adjustment of the suspender-lengths, complete reliance being placed upon the accuracy of the shop-work. The socketing procedure was substantially the same as that described for the cable-strands.

A novel feature, which may well be mentioned here, is the provision

of ladder-rungs attached to the suspender-ropes to give access to the main cables at each suspender-point, thus obviating the necessity

Figs. 23.



of handrails or hand-lines above the cables. The rungs (shown in *Figs. 23*) are round forgings, $\frac{11}{16}$ inch diameter, the ends of which

are upset and formed into slotted jaws which were squeezed and hammered on to the two parts of the suspender-rope after the latter were in place and carrying the full dead load. A total of four thousand, one hundred and fifty galvanized rungs was supplied.

Cable-Bands.

The suspenders are connected to the main cable by means of cable-bands of conventional design, as shown in *Figs. 23.* Each band consists of a pair of steel castings, shaped to the profile of the cable with internal fluting to fit on to the outer strands, the fluting being left rough-cast to aid the development of friction between the band and the cable. A vertical outer groove (the angle of which to the axis of the band varies to suit the varying slope of the cable) carries the suspender-rope, which is maintained in position by its own tension, and, during erection, by two small keeper-castings held by tap-bolts.

Each band is secured by four $1\frac{1}{2}$-inch high-tensile Mayari-steel bolts tightened to a specified tension of 33,000 lbs. per bolt, the total pressure on the cable being thus 132,000 lbs. at each band. At the steepest part of the cable this postulates a resistance to sliding of about 30 per cent. of the pressure. Tests made during the building of the Delaware River bridge indicate that a resistance to sliding of as much as 60 per cent. of the pressure is generated by a cable-band, the bulging of the cable outside the limits of the band being a large factor in the prevention of slip. Recent tests made in connection with the George Washington bridge have substantiated this result.

The ends of the cable-bands were counterbored to a depth of $\frac{1}{2}$ inch and, after the wrapping of the cable had been completed, the recesses were packed with lead wool in order to render waterproof the junction between the band and the adjacent wrapping. The longitudinal joints between the two halves of each band were sealed in a similar way.

Expansion-Joints.

The problem of expansion-joints for roadway-surfaces is more than ordinarily complicated in Canada, where the temperature range may often exceed 150° F., and where at the lower temperatures any moving parts are liable to become clogged by snow and ice with consequent local damage to the structure.

The usual method used where provision has to be made for a large temperature-movement is the use of expansion-fingers. A set of narrow steel bars, arranged like the teeth of a comb, is attached

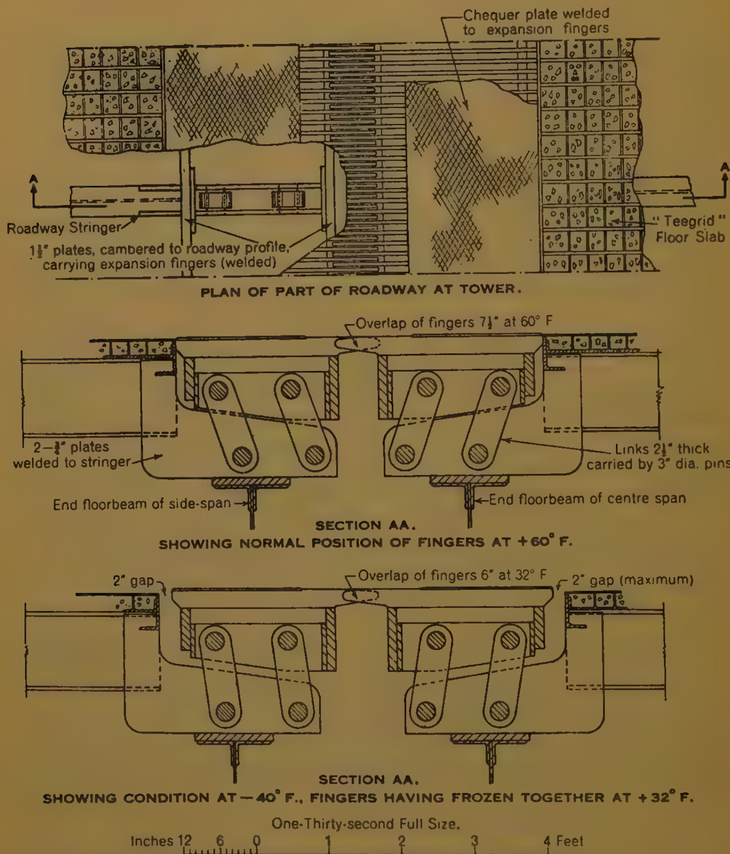
to the roadway on each side of the joint, so that the bars on one side slide in the spaces between those on the other. By this means the roadway-surface, except for narrow longitudinal slits, is continuous at all temperatures. The usual practice has been for each set of fingers to be solidly anchored to the roadway-slab. In the event of the small sliding clearances becoming frozen up, there has been the possibility of the fingers on one side being torn away from their anchorage in the slabs : this has occurred during recent cold winters on both the Quebec and Montreal cantilever bridges, and also on the Detroit—Windsor suspension bridge.

Expansion-joints of this type, but with an important alteration in the method of attachment, are provided for the roadway of the Island of Orleans bridge at the ends of the suspended spans. The total range of relative movement of the adjoining slabs at these points, due to various loading conditions together with a temperature range extending from $-40°$ F. to $120°$ F., is $11\frac{3}{16}$ inches at the main towers, and $5\frac{7}{16}$ inches at the cable-bents. A part of one of the joints at either end of the central span is shown in *Figs. 24*, p. 392. On each side of the joint the expansion-fingers (3 inches by $\frac{11}{16}$-inch flats, 2 feet $9\frac{1}{2}$ inches long, each set being spaced at $1\frac{11}{16}$-inch centres) are welded to the top surfaces (shaped to the roadway profile) of two $1\frac{1}{2}$-inch supporting plates, which extend across the full width of the roadway. These two plates, forming a unit with the fingers, are supported from each stringer by a pair of 12-inch links connected to extension-pieces welded to the stringer, so that the plates, and the expansion-fingers carried by them, are capable of a limited up-and-down parallel motion.

The links in their normal position are inclined from the vertical, so that each set of fingers, partly decked over with chequered plate, normally rests back against the adjacent roadway-slab, which is suitably stiffened with a heavy transverse angle to resist the resulting pressure. Under ordinary circumstances the links do not move, and relative movements of the floor-slabs are taken up by the sliding of the expansion-fingers past each other. In the case of the fingers becoming frozen together and the temperature thereafter continuing to fall, the two sets of fingers, locked together with ice, will rise owing to the movement of the links caused by the two floor-slabs retreating, and there will be small transverse gaps left in the roadway at the ends of the slabs. Usually, the weight of traffic or a rise in temperature will break the ice-bond and cause the fingers to drop back to their ordinary positions; should, however, this not take place at once, there is no danger of damage to the structure, and, if it is deemed necessary, the ice can be removed at the convenience of the maintenance engineer. A further improvement, recently

introduced with success on the three large bridges previously mentioned, was incorporated into the design by making the expansion fingers wedge-shaped in section in order that accumulations between them may more readily fall away. The fingers are planed from a

Figs. 24.



width of $\frac{11}{16}$ inch at the top surface to $\frac{3}{8}$ inch at the bottom. For the expansion joints in the sidewalks, the conventional sliding-plate arrangement is employed.

In view of possible variations from theoretical clearances arising during erection, the end roadway-stringers, fitted with special ends as described above, were fabricated a few inches longer than their theoretical lengths. They were then cut in the field to suit the actual normal distances (as erected) between the end floor-beams of the adjacent spans.

Lighting.

The roadway is electrically lighted throughout, lighting units of 400 candle-power being spaced about 150 feet apart on each side of the road. The lamps on the approach-spans are carried on the fence-posts, and those on the suspended spans are situated on the upper chords of the stiffening-trusses. Navigation-lights, as required by the Federal Department of Marine, are also provided.

Erection.

The six main piers for the suspension bridge were built during the open-water season of 1934, construction of the long approaches having taken place during the previous 3 years. Erection of the corresponding superstructure, a description of which is now given, began in the early autumn of 1934 and was completed in August, 1935.

Anchorages.

While discussion of the substructure does not come within the direct scope of this Paper, it is nevertheless desirable to make some reference to the method of construction of the anchor-piers, as this was influenced largely by the exigencies of the superstructure erection. These piers were built in six successive and distinct stages of construction, to three only of which reference will be made here.

In the first place, the walls of the base, 8 feet thick including the stone facing, were built to their finished elevation (above high-water level), with a heavy concrete sealing-layer covering the bottom of the excavation. This " cofferdam " stage provided a dry chamber in which the anchorage steel was erected and incorporated into the mass of the pier. The eight primary plates of each anchorage were assembled together as a unit in order to simplify adjustment, and adjustable holding-down bolts were utilized in order to facilitate accurate setting. The steel for these anchorages was the first part of the superstructure to be erected, the majority of the work being done in August, 1934. The mass of concrete used for the anchorages, extending the full length of the pier between walls, and consisting of some 760 cubic yards per pier, is indicated in *Fig. 10* (p. 374).

For the next principal stage of construction, it was required that the anchorage-steel should be easily accessible in order to attach the cables to it; that there should be space convenient for setting up the strand-hauling machinery and for storing the cable strands on the pier; and that it should be possible to erect the flanking approach-spans in order to give access to the suspended

spans for the erection of stiffening trusses and similar work during the winter months, when the use of floating equipment would not be possible.

Some thought was first given to the possibility of providing temporary steel towers to support the approach-spans from the partially-completed piers, but eventually the two contractors concerned decided, with the approval of the engineers, to advance the construction to the stage shown in *Figs. 10* and *11* (pp. 374–5). In this stage of construction, the pier-shaft supporting the side spans was complete, the anchorage-pits were easily accessible for the erection of the cables, and there was adequate room above high-water level for the establishment of hauling machinery and for the storage of the cable-strand reels. The walls of the piers being finished to their final elevation, there was no danger of flooding the anchorage-pits. Though the anchorage-piers in this condition presented a somewhat top-heavy appearance in end elevation, their strength and stability were entirely adequate for all loading contingencies that might arise. The piers remained in this state throughout the erection of the superstructure, the final stage of construction being completed in June, 1935.

It is of interest to note that a scale model in wood of an anchorage-pier was made by the engineers, in which blocks of wood represented the separate pours of concrete. This was found very useful by all parties concerned, and was constantly referred to by the Resident Engineer and by the contractor's engineers and foremen during construction.

Flanking Spans and Cable-Bents.

Erection of the 236-foot flanking spans and of the adjacent 150-foot approach-spans took place as soon as the concrete of the shafts of piers Nos. 15, 16, 22, and 23 was ready for service.

The 150-foot span between piers Nos. 14 and 15 was erected by cantilevering the trusses out as continuations of those of the existing 150-foot span on the shoreward side of the pier. For this purpose the upper chord-members were made continuous over pier No. 14 and the ends of the lower chords were suitably blocked in order to transmit the necessary compression. When the trusses were complete and ready to take their bearing on pier No. 15, the top chord sections were cut and the blockings removed, so that the trusses became simple spans. A 5-ton stiff-leg derrick with a 20-foot mast and a 50-foot boom was used to erect the 150-foot span.

The 236-foot span between piers Nos. 15 and 16 was erected by a similar operation of cantilevering, although intermediate temporary

support was provided by the use of timber towers. The cable-bent on pier No. 16 was then assembled and lashed to the 236-foot span. A 12-ton stiff-leg derrick with a 20-foot mast and a 70-foot boom was used for these operations.

A similar sequence of operations took place at the Island end of the bridge, all four approach-spans being erected between 27 August and 25 November, 1934.

Main Towers.

Prior to the erection of the towers, the concrete of piers Nos. 17 and 21 was very carefully dressed to the required level by bush-hammering and finishing with carborundum. A final checking with a long straight-edge showed the surfaces to be true and level, local variations not exceeding $\frac{1}{32}$ inch.

The welded steel pedestals and the bottom sections of the columns of each tower (these being the heaviest pieces, each weighing about 31 tons) were erected by a 50-ton floating crane, the property of the Quebec Harbour Commissioners. The remainder of the columns and bracing were erected by a creeper traveller designed for the purpose. This method of erection by creeper was selected as being the most expeditious and, consequently, that best adapted to avoid delays due to the bad weather expected at the site at this time of the year.

The creeper consisted essentially of a stiff-leg derrick, with a 30-foot mast and a 50-foot boom, mounted on a light steel platform built around the two columns of the tower and suitably braced to them. This platform was capable of being "jumped" to new locations as the tower reached various stages of erection. It was operated by two hoisting-engines located on the top of the pier, one of which operated the hoisting equipment and the other the jumping-tackle. The capacity of the creeper was 20 tons, the heaviest lift handled by it being a column section weighing about $18\frac{1}{2}$ tons. The creeper was itself assembled by a 20-ton floating crane. The creeper is shown (*Fig. 25*, facing p. 383) in use on the partially-erected tower on pier No. 21.

On account of the eccentric load due to the weight of the creeper, the derrick-mast of which was 8 feet off the centre of the tower, it was impossible to make any attempt at plumbing the towers during erection. Complete reliance was placed on the accuracy of the shop-work, material being drawn together at the column-splices so as to obtain as nearly perfect a bearing as possible. The accuracy of the machining of the splices was found to be such that the maximum space between the bearing surfaces at the splices was not more than a few thousandths of an inch. Before being dismantled,

the creeper was used to assemble a 5-ton guy-derrick (with 20-foot mast and 30-foot boom) on the top of the tower; this was to be used for the erection of the walkways and main cables.

After the towers had been fully riveted, the anchor-bolts tightened and the creeper dismantled, observations were made to determine the positions of the transverse centre-lines of the tops of the columns relative to the transverse centre-lines of the corresponding bases, and the deviations from plumb for the four columns were found to be $1\frac{1}{2}$ inch, $\frac{3}{4}$ inch, $\frac{7}{8}$ inch and $\frac{1}{2}$ inch respectively. That these amounts were so small is a tribute to the accuracy of the shop- and erection-work on the steel of the towers as well as to the care with which the concrete of the bases was finished.

Tower No. 17 was erected between 20 September and 17 October, and tower No. 21 between 26 October and 15 November, this work proceeding concurrently with the erection of the flanking approach-spans.

Temporary Walkways.

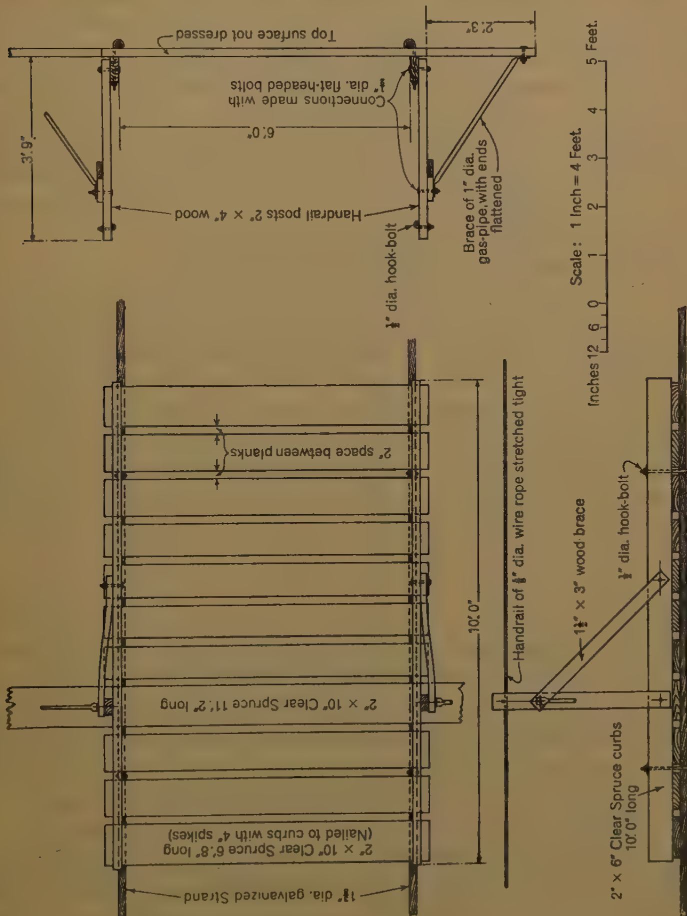
As soon as the erection of the towers and cable-bents was sufficiently far advanced, temporary suspended walkways were erected. The purpose of these was primarily to give access to the main cable for purposes of erection, adjustment, assembly of the cable-bands and suspenders, and erection of the stiffening trusses. Incidentally, in the earlier part of the winter when navigation was impossible on account of broken ice floating up and down the river with the tide, the walkways were the only means of access to the various towers and piers.

Each walkway (*Figs. 26*) consisted of a light deck of new clear spruce, 6 feet wide, supported on two $1\frac{3}{8}$-inch-diameter strands; these were carried on temporary saddles bolted to the towers and cable-bents, and attached to specially-provided anchorages by means of heavy turnbuckles. The total weight of each walkway was approximately 53 pounds per foot, and it was capable of carrying a uniform live load of from 50 to 60 pounds per foot without exceeding ordinary working stresses. The walkway-strands were of galvanized wire, made (for reasons given in the Appendix) to the same specifications as the main strands.

The walkways were erected as follows. After the strand-sockets had been connected to the anchorage at pier No. 15, the reels containing the walkway-strands were mounted on a scow which was towed across the river at high tide, the strands being hung on to the sides of the piers as the scow passed and being then attached to their anchorage at pier No. 23. This was done in late November,

1934, before the formation of heavy ice. The strands were then hoisted up and adjusted so that the appropriate marks (established during pre-stressing) coincided with the centre-lines of the saddles. The timber decks, including the handrail-posts, were made up at the site in units of about 10 feet in length. They were hoisted to the

Figs. 26.



Construction of Temporary Walkway.

tops of the towers and slid down the strands into position. The two walkways were connected at the middle of the central span by a light cross-bridge, which served also to brace the four walkway-strands together.

At the quarter-points of the central span there were further braces of 8-inch by 8-inch timbers secured beneath the four walkway-

strands. Similar braces were placed at the mid-points of the side spans. Subsequently, during a high wind, such violent torsional oscillation occurred in one of the side-span walkways that it was found necessary to stiffen them further by the addition of braces of the same kind at the quarter-points.

Vertical oscillation of the walkways, which would otherwise have been serious in the central span, was obviated to a great extent by a system of wind-guys under each walkway. Each system consisted of a $\frac{7}{8}$-inch wire rope (in the form of an inverted funicular polygon with a rise of about 90 feet), secured to the bases of the main towers and attached by vertical pairs of $\frac{3}{8}$-inch wire ropes to nine points on each walkway-strand. Tension in the $\frac{7}{8}$-inch rope was supplied by weights suspended from a tackle at one end. After the main cables were erected and adjusted, these wind-guys were removed and the walkways were lashed to the main cables.

The walkways were erected about 4 feet below the main cables, and, as the latter lengthened and sagged under the various additions of loading, this distance was maintained by adjusting the turnbuckles at the anchorages so as to lower the walkways. It was found that, owing to the open nature of the decking, deposits of snow were speedily removed by the prevailing strong winds, while formations of ice soon evaporated even at low temperatures. Erection of the walkways began on 16 November and they were ready for use on 12 December, 1934.

Main Cables.

The seventy-four cable-strands, each on a separate wooden reel, were shipped by water from Montreal to the site in November, 1934, and on arrival were stored on the base of anchor-pier No. 15. For erection, each reel was mounted in turn on bearings situated on pier No. 15, above and behind the anchorage. The outer socket was led off below the reel and was placed in a welded box, or " sleigh," of sufficient size to carry the socket, and shaped to slide on the deck of the walkway. Two strands, one up- and one down-stream, were hauled across simultaneously, the motive power being supplied through $\frac{5}{8}$-inch wire-rope hauling-lines by a double-drum hoisting-engine, situated on pier No. 23 behind the upstream anchorage. Horizontal hardwood rollers 6 inches in diameter were placed at the tops of the towers to prevent damage to the galvanizing of the strands, on account of the heavy bearing at these points.

On arrival at pier No. 23, the leading sockets were removed from the sleighs and attached to their appropriate eye-bolts at the anchorage; at the same time the inner sockets were removed from

the reels and attached to the anchorages on pier No. 15. In each case the sockets were set a few inches closer to the anchorage pins than the final positions, in order that any adjustment should be made by slackening the anchorage-bolts.

The sleighs were returned to pier No. 15 by $\frac{7}{16}$-inch wire-rope lines, pulled simultaneously by a double-drum hoisting engine located on that pier. The $\frac{5}{8}$-inch lines were hauled back with the sleighs. During the return journey of the sleighs the empty reels were removed from the bearings on pier No. 15 by the 12-ton derrick on the approach-span above, and full reels substituted. The time taken for the complete operation of hauling two strands across, securing them to the anchorages and returning the sleighs, was generally about 1 hour.

Each sleigh was always accompanied by a man, who guided it past obstacles and saw that no twisting of the strand took place. A telephone system was installed, communicating with the hoisting engines and with the tops of the towers, where watchmen were stationed. There was thus complete control of operations at all times.

Temporary splay-jigs of hardwood were mounted on the anchor-piers, the purpose of these being to constrain the cable to its finished shape during erection in order to facilitate the assembly of the final splay-casting. They were placed a few inches nearer to the anchorage than the position of the corresponding casting, and were so located as to hold the cable at its final normal elevation at the point in question. After each strand had been secured to both anchorages, it was placed by hand in the temporary splay-jigs and by hand into its appropriate grooves in the saddles of the cable-bents. It was lifted into the tower-saddles by means of the 5-ton guy-derricks on the tops of the towers, which were operated by hand winches, the marks on the strand being placed approximately on the centre-lines of the saddles. The strand was then ready for adjustment.

After final adjustment of the guide-strands, as described later, the remaining three strands of the first layer of each cable were placed in the saddles and adjusted in accordance with the position of the guide-strand. The same procedure was followed with the remainder of the thirty-seven strands for each cable, the strands of each layer being assembled on the saddles as soon as those of the previous layer had been adjusted. Work on the two cables was carried out concurrently.

On account of the bad weather and consequent delays experienced while the strands were being erected, it was found necessary during this period to prevent individual movement of the strands of the

incomplete cables. For this purpose, hardwood jigs were made, adaptable to constrain the strands to their proper positions in the cross section of the cable at successive stages of its development. These jigs prevented damage from chafing of the strands against each other, and were of considerable use in compacting the layers during adjustment. They were assembled at intervals of about 150 feet along the cables, and their use was continued until the erection of the cable-bands.

The first strand was unreeled on 13 December, 1934, and the cable assembly was completed on 29 January, 1935.

Adjustment of Cable-Strands.

Calculations were first made to determine the shape of the series of catenaries in which the free cable would hang, these calculations resolving themselves into the following problem: Given the lengths (obtained from the calculated dead-load lengths) of the unstressed strand from the anchorage-pin to the centre-line of the cable-bent, from the centre-line of the cable-bent to the centre-line of the tower, and between the centre-lines of the towers, it was required to find the various saddle-positions so that the sum of the horizontal projections of the five bights would agree with the horizontal distance between the anchorage-pins. It was a condition that the horizontal components of the stress in the strands should balance at the tower-saddles, and that, at the cable-bents (where the load, and consequently the friction, would be small, causing a tendency to slip during the initial stages of erection), there should be equal tensions at both sides of the saddle.

Commencing with an assumed value "c" for the horizontal component of the strand-tension for a strand of unit weight per foot, the lengths, sags, and horizontal projections of the catenaries were ascertained. The initial assumption was then altered in a series of approximations until the required result was obtained. For a temperature of $+15°$ F., chosen as the probable average temperature during the erection of the cables, these computations (shown in Table V) indicated that a free strand would tend to take up a position in which the centre-lines of the tower-saddles are deflected 1 foot $11\frac{5}{8}$ inches towards the shore, and those of the cable-bent saddles deflected $11\frac{3}{8}$ inches in the same direction. The symbols used in Table V are explained in *Fig. 27*, p. 402, and the catenarian properties used in its computation are set out in Table VI.

To facilitate the erection of the cable, the saddles had therefore to be offset by approximately the above distances. In order, however, to utilize the holding-down bolts, the tower-saddles were

TABLE V.

		Trial 1.	Trial 2.	Trial 3.
Centre span.	Assumed value for c	1400	1429	1434
	Strand length l *	1087·454	1087·450	1087·450
	$\frac{l}{2c} = \sinh\frac{K}{2c}$	0·3883764	0·3804930	0·3791667
	$\frac{K}{2c}$	0·3791996	0·3718629	0·3706234
	K	1061·759	1062·784	1062·948

[NOTE.—c has the same value both in the central and the side spans.]

		Trial 1.	Trial 2.	Trial 3.
Side spans	Strand length l *	439·823	439·821	439·821
	v	137·000	137·000	137·000
	$\sqrt{l^2 - v^2}$	417·942	417·940	417·940
	$\frac{\sqrt{l^2 - v^2}}{2c} = \sinh\frac{K}{2c}$ (1)	0·1492649	0·1462350	0·1457251
	$\frac{K}{2c}$	0·1487164	0·1457189	0·1452144
	K	416·406	416·465	416·475
	$\frac{v}{\sqrt{l^2-v^2}} = \sinh\frac{x_1+x_2}{2c}$ (2)	0·3277969	0·3277986	0·3277986
	$\frac{l}{\sqrt{l^2-v^2}} = \cosh\frac{x_1+x_2}{2c}$ (3)	1·0523549	1·0523557	1·0523557
	$\cosh\frac{K}{2c}$ (4)	1·0110790	1·0106357	1·0105621
	$(4)\times(3)-(2)\times(1)=\cosh\frac{x_1}{c}$	1·0150853	1·0156126	1·0157023
	$y_1 = c\cosh\frac{x_1}{c}$	1421·119	1451·310	1456·517

[NOTE.—The strand-tension being unchanged at the cable-bent, the quantity y_2 in the backstay has the value of y_1 in the side span.]

		Trial 1.	Trial 2.	Trial 3.
Back-stays	y_2	1421·119	1451·310	1456·517
	v	107·730	107·730	107·730
	y_1	1313·389	1343·580	1348·787
	Strand length l *	256·938	256·939	256·939
	$s = \frac{y_2^2 - y_1^2 - l^2}{2l}$	444·799	457·4546	459·6378
	$c = \sqrt{y_1^2 - s^2}$	1235·78	1263·306	1268·053
	$\sqrt{l^2 - v^2}$	233·262	233·264	233·264
	$\frac{\sqrt{l^2 - v^2}}{2c} = \sinh\frac{K}{2c}$	0·0943785	0·0923226	0·0919770
	$\frac{K}{2c}$	0·0942390	0·0921934	0·0918480
	K	232·917	232·936	232·936
	ΣK {required to be actual distance 2361·760 between anchorage pins.}	2360·405	2361·586	2361·770

* NOTE.—Strand lengths are computed between theoretical intersections at saddles. They are corrected for temperature (+ 15° F.) and for actual catenary tensions.

Lengths are in feet. Tensions are in lbs.

Fig. 27.

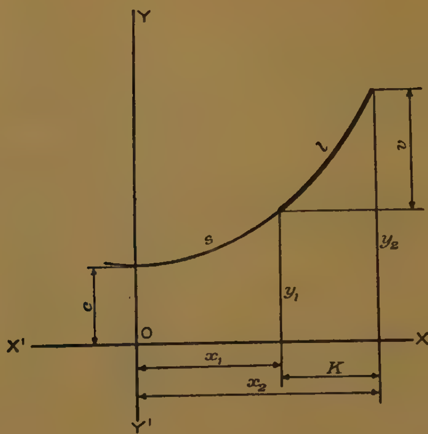


TABLE VI.

The equation to the catenary shown in *Fig. 27* is

$$\frac{d^2y}{dx^2} = \frac{1}{c}\sqrt{1 + \left(\frac{dy}{dx}\right)^2}$$

where c denotes $\frac{H}{w}$, or $\frac{\text{constant horizontal component of cable-tension}}{\text{weight of cable per unit length}}$

The following are properties of the catenary:

$$\frac{dy}{dx} = \frac{s}{c} = \sinh\frac{x}{c}$$

$$\frac{y}{c} = \cosh\frac{x}{c}$$

$$y^2 = s^2 + c^2$$

$$\frac{\sqrt{l^2 - v^2}}{2c} = \sinh\frac{K}{2c}$$

$$\frac{v}{\sqrt{l^2 - v^2}} = \sinh\frac{x_1 + x_2}{2c}$$

$$\frac{l}{\sqrt{l^2 - v^2}} = \cosh\frac{x_1 + x_2}{2c}$$

$$\cosh\frac{x_1}{c} = \cosh\frac{x_1 + x_2}{2c} \cdot \cosh\frac{K}{2c} - \sinh\frac{x_1 + x_2}{2c} \cdot \sinh\frac{K}{2c}$$

$$s = \frac{y_2^2 - y_1^2 - l^2}{2l}$$

offset 1 foot 9 inches. The cable-bent posts were inclined so that the saddles were 10 inches out of plumb (this was calculated to be their natural position when the tower-saddles were constrained as

Fig. 28.

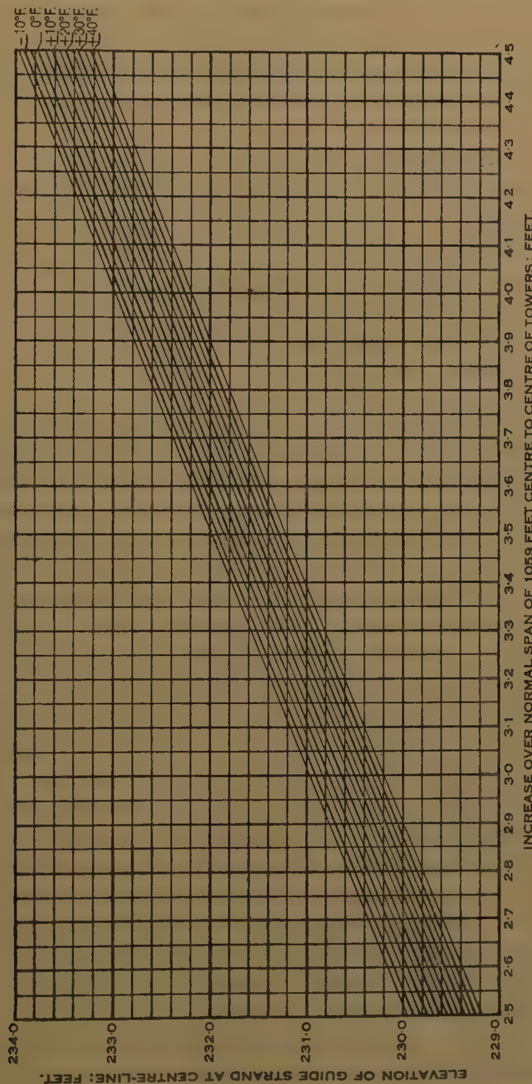


CHART FOR ADJUSTMENT OF GUIDE-STRAND IN CENTRAL SPAN.

above) and were rigidly secured to the approach-trusses by means of 12 inches by 12 inches timber shores and wire-rope lashings with turnbuckles.

A chart was prepared (*Fig. 28*) for the central span, showing the

required elevation of the guide-strand at the centre of the span for various temperatures and horizontal spans. For this purpose, calculations, similar to those previously described, were made for temperatures of $-15°$ F. and $+45°$ F., intermediate points in the curves being interpolated. By the use of this chart it was possible to adjust the guide-strand in the central span independently of its position in the other spans. Similar charts were made for the side spans and the backstays.

The two guide-strands were then erected as previously described, and the following data were obtained for their adjustment:

(*a*) The sags of the five catenaries relative to the catenary-chords.

(*b*) The positions of the centre-lines of the four saddles relative to their final normal positions.

(*c*) The positions of the two ends of the strand relative to the anchorage-pins.

The elevation of the strand at the middle of the central span was obtained by means of a level set up on a platform on the cross-bracing of tower No. 17; this was sighted on a target on tower No. 21 set at the same elevation as the telescope, and the reading was taken on an inverted rod held with its base level with the centre-line of the strand.

For the sag of the side spans, a transit was set up at a known point near the end of the 236-foot approach-span. The telescope was directed at a target painted at a given location near the top of the tower, and was then rotated on its vertical axis until the horizontal cross-hair intersected a rod held vertically on the point of maximum sag of the strand. The elevation of the strand at the point of maximum sag in the backstay was obtained by direct vertical measurement from a point of known elevation on a timber, which was secured to the 236-foot approach-span truss.

For observing the position of the top of a tower, a mining transit, equipped with a side telescope with prismatic eye-piece so that the necessary nearly-vertical readings could be made, was used. This instrument was set up on the pier at the foot of the tower, with the plumb-bob over the longitudinal centre-line of the bridge. A sight was first taken to a target on the centre-line of the other main pier, and the telescope was turned through 90° on its vertical axis. Then, using the horizontal axis, readings were taken on two level-rods projecting horizontally at the tops of the two posts.

The accurate setting of the guide-strands being of paramount importance, great care was taken in making these observations. It was necessary that all the observations should be made as nearly

simultaneously as possible, and during a period of suitable weather-conditions, that is, a period without wind and sun and during which there would be very little variation of temperature. As there was almost incessant wind at the site, these conditions were difficult to fulfil, and considerable delay was experienced on this account. The guide strands were in place on the saddles on 14 December, 1934, but final adjustment observations were not made until 3 January, 1935.

Observations were commenced early in the morning, as soon as there was sufficient light, this being the time of day when there was the greatest likelihood of a calm spell. The time required to take a complete set of readings (each observation being made several times, and, whenever possible, by different observers) was about three hours, and on several occasions the work had to be abandoned on account of unexpected high wind, snow, or sunshine. To make the work as speedy as possible, four transits and one level were used, and these were kept stored near the points where they were set up. Six observers shared the work. The frequent occurrence of sub-zero temperatures made it advisable to protect the observers from wind, which greatly aggravated the cold. A solid timber wind-shield was therefore built around each instrument position. These, besides making it possible to use the naked hands for a longer period without excessive discomfort, gave the observer a sense of security when working in an otherwise precarious situation.

After a set of observations had been taken, the results were tabulated, and the actual strand-elevations at the points of maximum sag were compared with the required elevations obtained from the charts (*Fig. 28*). The strands were then moved at the saddles and anchorages by the small amounts necessary in order to produce the requisite alterations in sag in the various spans; these movements were made as described later. After the first adjustment, two more sets of observations were made, followed by a final adjustment.

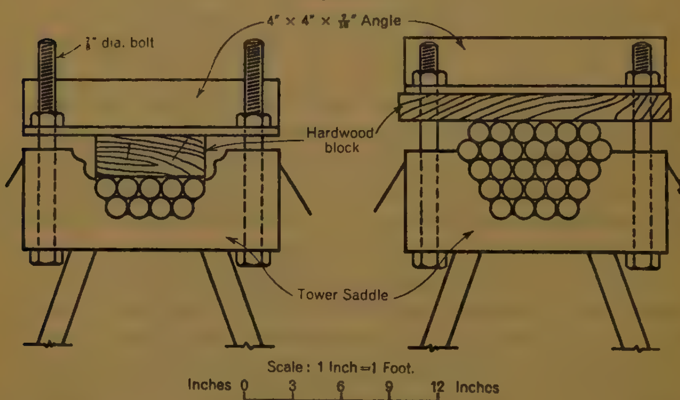
The remaining strands were adjusted individually to their appropriate positions in relation to the guide-strand. The starting-point for adjustment was the saddle on tower No. 21, on which the strands were located by means of the painted reference marks and secured against further movement by means of hardwood blocks bolted down on to the strands as shown in *Figs. 29* (p. 406).

The first adjustment of a strand was made in the central span. During a period of calm weather, preferably without sunshine, the required increase of sag was measured with a foot-rule. The ratio $\dfrac{\text{increase in strand-length}}{\text{increase in sag}}$ had previously been computed for small movements of the strand at the saddle, and was found to be very

nearly one-half. The strand was therefore moved at the saddle of tower No. 17 a distance equal to one-half of the required increase in sag. The movement of the individual strand by itself was accomplished with a ratchet turnbuckle, pulling by means of a clamp made of two plates bolted together to grip the strand. It was found that if the wooden-block clamp was kept tightly bolted down, the movement of any one strand could be easily effected by the turnbuckle without disturbance of the other strands.

It was found convenient to make the adjustments of a strand in the side span, and the corresponding backstay, simultaneously. From the vertical movements required at the central points of these spans, the amount by which the anchor-bolt should be slackened

Figs. 29.



was computed, the ratios $\dfrac{\text{movement at saddle or anchorage}}{\text{corresponding change in sag}}$ for the side spans and backstays being respectively one-sixth and one-fourteenth. The slackening of the nut was accomplished while the bolt was pulled back shorewards a few inches by means of a hoist-line attached to a special nut. Movement was then made at the cable-bent saddle to produce final adjustment in the side span, after which any further adjustment required in the backstay was made by further motion of the anchor-nut.

The degree of adjustment achieved was most satisfactory. In spite of adverse weather-conditions, the strands were adjusted to within ½ inch of their required sags, and, in most cases much more closely than this. The extension of the cable in the central span being about 2 feet under dead load, such small variations are seen to be of no account.

It was interesting to note the effect of sunshine on the cable. The strands on the south side would become a few degrees higher in temperature than the remainder, with the result that the cable would twist at the span-centres until the top layer of strands was as much as $\frac{1}{2}$ inch low on the south side. With the disappearance of the sun the cable would right itself. In the final state of construction, this phenomenon ceased to occur on account of the constraint afforded by the loads on the suspenders.

It will be noted from the foregoing description that the reference-marks, of which several were made on the guide-strands during pre-stressing (see Appendix), were used only as a rough guide in the field, the guide-strands being set in accordance with calculated sags, and the remaining strands being adjusted to the guide-strands. After assembly of the cable, however, it was found that the marks on the strands so nearly coincided with the centre-lines of the saddles that it is questionable whether any significant gain in accuracy was achieved by the procedure of adjustment by observations on sags and tower movements.

The Author inclines to the opinion that, given pre-stressing and pre-measurement of the high standard obtained here, and provided that the steelwork of the towers, bents, and anchorages is accurately located, the procedure of guide-strand adjustment could be eliminated in this type of construction.

Cable-Bands and Suspenders.

The cable-bands were unloaded in the contractor's yard and hauled over the ice by horse-sleighs to the main piers, whence they were hoisted to the tops of the towers. They were then skidded down the walkways and assembled on the cable by hand, care being taken to ensure each bolt was fully tightened. During the progress of erection of the trusses and floor, the tightness of the bolts was checked several times, a final tightening being performed after the cables had received their full dead loads.

The assembly of the bands, after delivery at the main piers, occupied two working days. Owing, however, to the internal dimensions of the castings having slightly overrun, it was possible in the first case to tighten some of them so that the two halves were in contact. To obviate any danger of the bands slipping on the cable every band was removed, and $\frac{3}{16}$ inch was planed off each casting. The bands were removed in rotation, so that there was always a sufficient number in place to preserve the shape of the cables. This caused a delay of 2 or 3 days.

The suspender-ropes were taken up to the tops of the towers in a

similar manner and were erected by hand. To ensure that equal tensions would obtain in the two parts of each rope, the centre-mark (established during pre-stressing) was carefully centred over the top joint of the cable-band and kept in position by the keeper castings shown in *Figs. 23.* Special precaution was taken at all times to avoid damage to the galvanizing of the ropes and cables. The suspender-ropes were erected in 3 days, the assembly being complete on 9 February, 1935.

Stiffening-Trusses.

The erection of the trusses was a comparatively straightforward process, the ice on the river forming a very convenient means of transport from the railway to the site. The contractor established a "yard" on the railway at Montmorency Village, and, in order to save congestion (there being in all over a hundred truss sections weighing from 5 to 9 tons each), consignments were arranged to arrive only as they were needed. The sections were unloaded from the railway by a derrick-car and stacked on timbers on the snow. Each section was hauled over the ice on sleds by a motor tractor and unloaded immediately below its final position on the bridge, while the floorbeams and lighter members were hauled by teams of horses. The ice was, in general, about 2 feet thick and exhibited no signs of yielding under the loads, the heaviest of which weighed about 15 tons.

The erection of the trusses was started in the middle of the central span. Two adjacent sections of one side were bolted together and raised as a unit, in order to make connection with two suspender-ropes (each individual truss-section having only one suspender-connection). Two sets of falls were used on each side of the bridge, and these were hung from points on the main cables, which were suitably protected for the purpose. The lifting effort was supplied by the hoisting engines on piers Nos. 17 and 21, the hauling-lines running up the towers and along on top of the walkways to the lifting-tackles. Both engines were operated simultaneously and could be used to raise trusses on either the upstream or the down-stream side as required. After the first two sections of each side were in place, further sections were lifted one by one and each was secured by connecting it to a suspender-rope and to the already-erected adjacent section. The trusses as erected were bolted into lengths consisting of four or five sections, each section being at first connected to the adjoining sections by a single bolt at the top-chord joint only, the deflection of the cables making complete connection impossible. The bolting of the joints was accomplished as soon as

Fig. 30.

ERECTION OF FIRST STIFFENING-TRUSS.

Fig. 31.

APPLICATION OF CABLE-WRAPPING.

Fig. 32

THE COMPLETED BRIDGE.

sufficient dead load was in position, and they were riveted as soon as the abutting surfaces could conveniently be brought to bear on one another. It was, however, impossible to complete the riveting of the top-chord joints until the full dead load, including the floor-concrete, was in place. No attempt was made to adjust the truss joints in any way, full reliance being placed on the accuracy of the shop-work to ensure that the anticipated stresses would develop.

Erection proceeded concurrently in both side spans, commencing at the cable-bents. The trusses were at first lifted from the ice by 5-ton derricks on the approach-spans, these being moved forward on to the suspended side-span floors as erection proceeded. After the erection of the first few sections the use of the derricks was discontinued in favour of erection by the method used for the centre-span trusses, that is, by hoisting the truss-sections on tackles attached to the main cables. Assembly of the trusses began on 11 February and was finished on 25 February, 1935. The erection of the first truss-section is shown in *Fig. 30* (facing p. 408).

At all times during the erection of the trusses, careful watch was kept on the movement of the tops of the main towers, and at no time was the tower-deflection permitted to exceed about 1 foot horizontally. These deflections were controlled at first by jacking the tower-saddles from their offset positions to their final central positions, and later by concentrating the erection of the trusses on either the central span or the side spans as necessary.

Tower-Saddles.

No difficulty was experienced in jacking the tower-saddles. The undersides had been heavily coated with tallow when they were first erected and the jacking was easily accomplished with two 25-ton jacks at each saddle, thrusting against a temporary steel jacking-bracket bolted to the tower. The full movement of 1 foot 9 inches was obtained in two operations, the saddles being first moved 9 inches after five truss sections had been erected on each side in the middle of the central span; the truss-weight on each side was then 88,300 lbs., and the tower-posts were inclined an average of $8\frac{3}{4}$ inches towards the river. After jacking, the tower-posts were found to be inclined an average of 2 inches towards the shore. The remaining 12 inches movement was made when approximately one-half of the stiffening trusses had been erected. Before jacking, the tower-posts were leaning about 4 inches towards the river, and after jacking the average inclination was $6\frac{1}{4}$ inches towards the shore. The two saddles of one tower were always jacked simultaneously, to obviate distortion of the tower-bracing.

Floor-Grids and Fences.

The sections of " Teegrid " and " Anglgrid " and the fence-panels were also erected from the ice. This work commenced on the 26th February, 1935, and the total time taken for assembly and welding was eight weeks. In this connection 1,450 lbs. of welding rod were used, the actual time of welding being 645 man-hours.

The floor-grids were filled with a 1-to-2·2-to-3·5 concrete mixture, with aggregate graded up to ½-inch stones, 4½ gallons of water being used per sack of 87½ lbs. of cement; this gave a slump of about 1½ inch. The concrete was vibrated by means of two spade-vibrators attached to a 2-inch by 12-inch plank laid athwart the roadway. The floor-grids on the suspended spans were concreted during one week's work (20 to 27 May, 1935).

Cable-Wrapping.

Before wrapping, the cables were treated with a thick coat of red-lead paste, after which the wood fillers (see Fig. 7, Plate 1) were placed in position. The wrapping-wire was applied by a machine (*Fig. 31*, facing p. 408) consisting essentially of a revolving frame carrying three coils, from which three adjacent turns of wire were placed simultaneously. The frame was driven by a reciprocating compressed-air motor constrained to travel above and parallel with the cable by a beam clamped to the cable. The whole machine was propelled along the cable in an upward direction by the pressure exerted by the wrapping wire as it was laid against that already in place. At the beginning and end of each section of cable (bounded by cable-bands or saddles) the wrapping was applied by hand, the ends of the wires being soldered into place. A further coat of red-lead paste was applied to the cable afterwards. Final painting consisted of two field coats as for the trusses and towers.

The maximum speed of machine-wrapping was 18 inches per minute, the average speed being about one-half of this rate. The total time spent on wrapping was 5,650 man-hours, this work being done between the dates 26 April to 1 May and 31 May to 15 August, 1935.

Painting.

All steel surfaces received one shop-coat of red-lead paint, particular treatment being given to surfaces inaccessible after assembly. Two field coats, of battleship-grey and grey-green respectively, were applied after erection.

A view of the finished bridge is shown in *Fig. 32* (facing p. 409).

General Information.

The total weight of steel in the superstructure of the central portion of the Island of Orleans bridge is made up as follows. The quantities below are obtained from the actual shipping weights, and are given to the nearest ton.

Anchorages	114	short tons.*
Cable-bents (with saddles)	47	,,
Main towers (with saddles)	630	,,
Main cables (including wrapping-wire)	419	,,
Cable-bands, flashings, etc.	17	,,
Suspender-ropes (with sockets and rungs)	25	,,
Stiffening-trusses (with laterals)	864	,,
Floor-framing for suspended spans	518	,,
Floor-grids for suspended spans	518	,,
236-foot approach spans complete	463	,,
Total weight of steelwork	3,615	,,

The contract relating to the manufacture of the superstructure contained a clause whereby the contractor was required to give first preference to materials of Canadian production or manufacture, and, when these were not obtainable, to give preference to those of British production or manufacture.

Of the total weight of steel ordered by the fabricators, the following were the percentages according to the country of origin.

Canada	66·7	per cent.
Great Britain	22·9	,, ,,
U.S.A. (wire for cable-strands)	10·4	,, ,,

The total cost of the bridge was about $3,500,000.00 (£700,000 at current rate of exchange, which is $4·98). The cost of the central portion described in this Paper, consisting of the suspension superstructure, the six main piers and the two 236-foot flanking-spans, was made up of the following items.

The contract for the construction of the substructure (being the six main piers) was let to the Foundation Company of Canada, Limited, the price being $461,403.00 (£92,281). The cement and the cut-stone for the piers were supplied separately from this contract, the stone costing $98,000.00 (£19,600) and the cement $51,537.00 (£10,307).

The Dominion Bridge Company, Limited, were awarded the contract for the fabrication and erection of the superstructure

* See note on p. 360.

(including the steel floor-grids) at a lump-sum price of \$813,747.00 (£162,749). As a special condition of the contract, the fabrication of the two 236-foot flanking spans was carried out by the McKinnon Steel Corporation, Limited. The work of making the cable-strands and suspender-ropes was sub-let to the Dominion Wire Rope Company, Limited. The wire for the cable-strands was made by the American Steel and Wire Company, Ltd., and that for the suspender-ropes by the Steel Company of Canada, Limited.

Concrete for the "Teegrid" floor, for the deck-slab on the 236-foot approach spans, and for a 1½-inch wearing surface throughout the approach-spans, was supplied and placed at a contract price of \$24,500.00 (£4,900).

The contract price for the electric lighting for the whole bridge was \$18,803.00 (£3,761).

The plans for the location of the bridge were prepared by the staff of the Department of Public Works, Province of Quebec, under the Chief Engineer, Mr. O. Desjardins, who also superintended the design and erection of the approach-roads and viaducts. The Consulting Engineers for the central portion were Messrs. Monsarrat & Pratley of Montreal, who were responsible for the design, fabrication and erection of this part of the project. The Author was employed by the Consulting Engineers, to whom he is indebted for permission to present this Paper, in the capacity of Assistant Engineer in the design of the suspension bridge and in the fabrication and erection of the cables and suspenders.

In conclusion, the Author wishes to place on record his thanks to Mr. P. L. Pratley, M.Eng., M. Inst. C.E., for valuable criticisms offered during the preparation of this Paper and for having given facilities for reference to the drawings and records. The Author is also indebted to the Dominion Bridge Company, Limited, for access to several drawings and photographs.

The Paper is accompanied by twenty-seven sheets of drawings and twenty photographs, from some of which Plate 1, the Figures in the text and the three half-tone pages have been prepared, and by the following Appendix.

APPENDIX.

The Pre-Stressing of the Cable-Strands and Suspender-Ropes.

General Notes.

The physical properties of a steel wire strand or rope depend upon many factors, among which may be cited the quality and uniformity of the original wire, the judgment and accuracy used in calculating wire-diameters and in selecting lengths of lay, the methods and machinery employed in the actual construction, and the skill and care of the workmen who carry out the process.

A new wire rope, when first subjected to tension, will generally be found to be somewhat irregular in its elastic characteristics. These become more uniform and definite after a period of use, during which period the modulus of elasticity, at first considerably lower in value than that of the individual wires used in the rope construction, increases as the lay of the rope becomes consolidated; and at the same time a permanent "structural" lengthening (as distinct from elastic lengthening, which is proportional to the tension and which disappears when the latter is removed) takes place.

In the case of wire rope or rope-strand for use in a permanent structure (as, for example, the counterweight-ropes of a lift span or the main cables or suspenders of a suspension bridge), however, it is of great importance that uniformity of elastic properties should obtain before the rope or strand is put into use. A high modulus of elasticity, with consequent smaller deformations under stress, is advantageous, while uniformity of modulus ensures that each strand or rope takes its proper share of the load. The manufacturing tension has a direct effect upon the elastic properties of new rope, and the uniformity of these properties can to a great extent be assured if this tension is comparable with the future working-load of the rope. Such manufacturing tension is, however, generally impracticable in the fabrication of larger ropes and rope-strands, and it is therefore necessary to produce the required effect by means of pre-stressing.

This operation of pre-stressing consists in the application of tension to an extent, and for a period, adequate to preclude the possibility of subsequent "structural" or permanent lengthening (with consequent change in modulus of elasticity) within the range of working-loads, and at the same time to produce a high and definite value of the elastic modulus.

The following important incidental advantages are also derived from the pre-stressing process. The rope or strand may, after pre-stressing, be placed under a suitable tension and its length accurately measured and marked. A thorough inspection of the rope can be made at the same time, during which any defects in the galvanizing or in the lay of the rope may be noted. Such defects may then, if advisable, be rectified before shipping to the site. In addition to this, the process of pre-stressing to a tension beyond the working tension of the rope is in itself an excellent practical demonstration of its capability to withstand the latter load in safety.

Specifications.

The Engineers' specification relating to cable-strands contained the following clauses:

> "PRE-STRESSING.—Each $1\frac{3}{8}$-inch strand shall be finished to a length in excess of the final length of the strand in place and then pre-stressed to a tension of at least one-half of the specified ultimate strength of the strand and held at this tension for 30 minutes. The tension shall then be decreased to 60,000 pounds at which load the strand shall be measured and the necessary reference points established for use during erection. . . . In addition, tests for modulus of elasticity shall be made on long lengths of each strand during the pre-stressing process. . . ."
>
> "TOLERANCE IN LENGTH.—The measured lengths shall not vary by more than 1 inch, up or down, from the specified lengths."

Plant and Equipment.

In order to fill these requirements, the contractor decided to construct what is the first and only pre-stressing plant in the Dominion. In view of the very satisfactory working of this plant and the good results obtained from it, it merits description.

The site selected was a level piece of land near the village of Longueuil, on the south side of the St. Lawrence River, near Montreal. It is about 12 miles from the contractors' works in Lachine and a similar distance from the works (also in Lachine) of the sub-contractor who manufactured the strands and ropes for the suspension bridge. This location was chosen on account of the facilities for shop-work provided by a commodious steel-fabricating plant belonging to the contractor.

The construction of the pre-stressing plant took place during the winter of 1933, and it is interesting to note that it was first used for the counterweight-ropes of the recently reconstructed Second Narrows bridge at Vancouver, B.C., the pre-stressing of these ropes being called for by the same consulting engineers. The experience gained during this work was most valuable, and led to several improvements being made in the plant itself. For the purpose of further experiments, the four temporary footwalk-strands for use during the erection of the Island of Orleans bridge were made to exactly the same specification as the seventy-four permanent cable-strands, and were pre-stressed before the latter.

The layout of the pre-stressing plant is shown in Figs. 33, Plate 2. Essentially it consisted of two heavy concrete anchor-blocks, about 2,500 feet apart, between which the cable strand was stretched at full length. Between these anchorages the strand was carried on a stout wooden bench or trough 1 foot 6 inches wide and 2 feet above a wooden walkway which ran on the ground alongside. A light 2-foot 6-inch gauge railway-track also ran between the two ends of the plant, with a spur line giving access to the shops.

Tension was applied at the south end of the plant by a hydraulic ram of 75 tons capacity, with a 12-foot stroke and a piston area of 98 square inches, served by two electrically-driven reciprocating pumps. The strand-socket was connected to a series of links, adjustable so that different lengths of rope could be handled. A further system of links made it possible to stretch the rope to a maximum extent of 23 feet by utilizing two strokes of the ram. At the north end, the rope-socket was attached to a link connected to a steel anchorage, which was secured to the concrete of the foundation. The machinery for reeling and unreeling the ropes was also situated at this end, and was driven

by a 15-HP. electric motor. The machinery at each end was housed in a steel-framed wooden building.

Apart from the experimental work, most of the pre-stressing was done at night, commencing usually at 8 p.m. and finishing at 6 a.m. This was in order to make use of the more uniform and constant temperatures then prevailing, the long strands being very sensitive to variations in temperature.

A system of electric lighting was installed, with powerful lighting-units suspended from 15-foot poles placed at 120-foot intervals. These poles also carried a power-line serving the machinery at the two ends of the plant. There was also telephonic communication between the two ends, installed primarily for use in controlling reeling operations and later used principally by the operators of the extensometer-gauges in correlating their observations. Suitable shelters were provided, so that work could be carried out in all weathers, the only trouble experienced from rain being some personal discomfort and a certain amount of difficulty in controlling the reeling and unreeling operations, on account of the slipperiness of the trough. The contractor's financial outlay for the construction of the pre-stressing plant was about $28,000.00 (£5,600 at current rate of exchange).

Preliminary Work.

The counterweight-ropes of the Second Narrows bridge were of regular lay, 1¾ inch in diameter and 1·248 square inch in metallic area, built up of six strands of nineteen wires each around a hemp centre, their ultimate strength being about 265,000 lbs. They were delivered in lengths of 2,405 feet, a make-up length of wire rope being used for pre-stressing. The pre-stressing load was 52,000 lbs., and the modulus of elasticity after 30 minutes of this tension increased on the average from about 16,000,000 lbs. per square inch to about 19,000,000 lbs. per square inch. Each length was marked for cutting while stressed with a tension of 25,000 lbs., this approximating to the working load of the ropes. At the same time, a reference-line was painted along the top of the full length of the rope, in order to show up any twisting which might occur during the releasing of the load, or during either the socketing or the erection. It may be noted here that the effect of a complete twist in one of the counterweight ropes (180 feet in length) was an alteration in length of about 1 inch.

The general procedure for pre-stressing being outlined later, a description will be given here of those features only which were found to be unsatisfactory and in which improvements were made subsequently.

During the earlier work, the applied tension was measured by a gauge giving the oil-pressure on the ram in the cylinder. Observations on the extension of the rope showed excessive variations in the stress-strain ratio, and it was recognized that these discrepancies were due to internal friction between the ram and the cylinder. It was found that, during the application of load, the ram came to rest before the tension indicated by the pressure-gauge had been fully transmitted to the rope; and in the same way, during reduction of load, the tension in the rope exceeded that indicated by the pressure-gauge. The maximum difference between the indicated and actual tension was found to vary, with a maximum value of about 8,000 lbs.

Extensometer-Gauges.

This method of measuring the load was evidently not sufficiently accurate for the measurement of the cable-strands, where a variation in load of 1,000 lbs. caused a change in length of about 1 inch. The measuring device

which was finally evolved was an extensometer-gauge consisting of a calibrated steel bar, the tension in which was indicated by very accurate extensometer readings. This apparatus is shown in *Figs. 34.* A round bar having a cross-sectional area of 2 square inches (1·595 inch diameter), and 4 feet 8 inches in overall length, was upset at either end to take a 2-inch screw-thread, and was incorporated into the tension system between the straining-head of the ram and the end of the cable-strand. The bar was made of heat-treated special steel, with an ultimate strength of 135,000 lbs. per square inch and yield-point at 114,000 lbs. per square inch. The gauge-length utilized was 30 inches, at each end of which a cross-head was secured by two hardened steel points, which were screwed into the bar diametrically opposite to one another. Two gauge-rods, fastened to one cross-head and sliding on supporting rollers in the other, carried dial gauges which, operated by plunger-rods, registered relative movements of the cross-heads in ten-thousandths of an inch. By taking the average of the readings of the two dials, giving the average extension of the two sides of the bar, any misleading effects due to bending stresses were eliminated. The extensometer gauge was calibrated for loads up to 160,000 lbs. in a 300,000-lb. Olsen testing machine, which was itself calibrated by means of an Amsler block. The results of calibration, which were uniform and consistent throughout several applications of load, were tabulated for use in the pre-stressing plant. An average needle-movement of one division of the dial (that is, $\frac{1}{10,000}$ inch) corresponded closely with a change of 200 lbs. in the load.

Two such extensometer-gauges were installed, one at each end of the pre-stressing plant. The average of the loads indicated at the ends was considered to be the average load in the strand under tension.

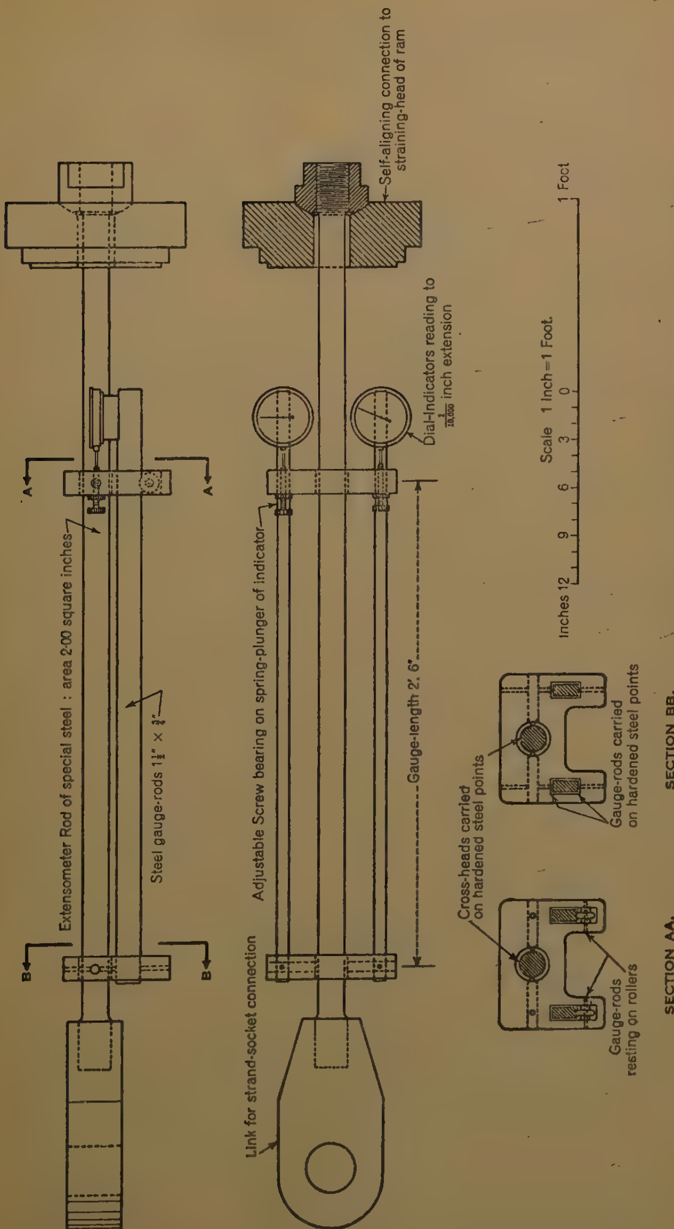
Further Preliminary Work.

During the pre-stressing of the temporary footwalk-strands, it was found that the tensions recorded by the two extensometer gauges differed by amounts varying up to about 5,000 lbs., this difference being due to friction generated between the strand and the wooden trough. To obviate this, rollers made from 1¾-inch diameter pipes mounted on ball-bearings were placed at intervals of about 50 feet along the trough, so that the strand rested entirely on these when it was in a state of tension. The friction, as given by the difference of the two gauge-readings, was decreased to a maximum of about 2,000 lbs. by the use of these rollers.

At the same time it was noticed that, while it was possible by careful regulation of the fluid-pressure of the ram to hold the rope at a given extension for a short time, there was a tendency to "creep" when any attempt was made to hold it during longer periods. A locking-device was installed, consisting of an adjustable anchor-block attached to the fixed cylinder of the ram, to which the straining links (which had pin-holes at 12-inch intervals) could be pinned at any point of their run. This ensured that the length of the strand would remain unchanged during any period of time, such as might be required for measuring it and establishing marks on it for use during its erection.

Some difficulty was also experienced on account of the tendency of the rope to twist under load, and thus to overturn the trolley supporting the sockets of the rope and of the make-up piece. This trouble was removed by replacing the make-up rope with a system of adjustable links heavy enough to prevent the trolley from overturning. Rotation of the socket at the fixed end was prevented by the provision of guides for the ends of the pin which connected the socket to the extensometer-link. It may be noted here that the cable-

Fig. 34.



EXTENSOMETER-GAUGE FOR MEASURING PRE-STRESSING LOADS.

strands for the suspension bridge, on account of their construction (which consisted of two opposite lays), exhibited very little tendency to twist when they were under load; the suspender-ropes, however, were more susceptible to twisting on account of their single-lay construction.

Cable-Strands.

The procedure carried out for the cable-strands was as follows. There being seventy-four cable-strands to be pre-stressed, measured and marked, six marking stations were established along the trough. Concrete monuments were built, each one being fitted with a steel plate at the level of the trough. Very careful measurements were made at night, during periods of uniform temperature, and lines were scribed on the plates at the distances required. The points marked on each strand were those corresponding with the centre-lines of the towers and cable-bents, the centre-point of the bridge, and the cutting point for the final outer socket. On four of the strands, the calculated positions of the suspender-ropes were also marked. Exceptional care was taken in the marking of these latter, and of the two guide-strands.

The strands were each delivered on a stout wooden reel of 4 feet internal diameter and weighing about 900 lbs. Each strand had a permanent socket on the inside of the reel. The outer socket was a temporary one (neither tinned nor grooved), which, after pre-stressing, was cut off and, after being cleaned, grooved and tinned, replaced as a permanent fitting. The delivered lengths of the strands ranged from 2,495 feet to 2,505 feet, the excess above the final length of 2,468 feet being for use in tests. Each reel in turn was fitted, in the shops adjoining the pre-stressing plant, with a steel shaft and chain drive; it was then placed on bearings mounted on a four-wheeled truck, which ran on the narrow-gauge track and was drawn by a petrol locomotive to the north end of the plant. Then, by means of a turntable, it was placed in position in front of the anchorage and by the side of the reeling-mechanism. The reel remained on the truck, which was jacked up and its weight transferred from the wheels to a solid foundation. The outer socket was led off from under the reel and attached by a pin to a small two-wheeled trolley running on rubber tires in the cable-stretching trough. The trolley was pulled along by a $\frac{7}{16}$-inch wire rope running around a sheave at the south end and hauled on to a 10-inch drum at the north end. The rate of unreeling was controlled by the speed of the motor and by a mechanical brake acting on the shaft of the reel. On arrival at the south end, the socket, still supported on the trolley, was secured to the make-up links, which were adjusted to suit the strand length, the $\frac{7}{16}$-inch line being disconnected. The other socket was removed from the interior of the reel and fastened to the north anchorage. The reel stayed in place on the truck, the rope lying clear beneath it.

To rewind the rope on to the reel, the procedure was almost the reverse of that outlined above. The inner (and permanent) socket was replaced in the interior of the reel, with which the winding gear was connected by a clutch which simultaneously released the drum carrying the $\frac{7}{16}$-inch line. The strand was then hauled back by the rotation of the reel, its own friction on the wooden trough giving the tension necessary for reeling. The trolley, carrying the outer socket, was drawn back at the same time, pulling with it the $\frac{7}{16}$-inch line in readiness for use in the next unreeling operation.

An initial tension of 10,000 lbs. was applied to straighten out the strand (the extensometer-gauges having in the first place been carefully adjusted to read zero) and at this load a reference-mark was established on the strand at

2,400 feet from the dead end. The load was then increased until the average of the two gauge readings indicated the pre-stressing load of 120,000 lbs., the 2,400-foot mark meanwhile moving about $10\frac{1}{2}$ feet. At this load the straining-links were locked and the pressure was released from the ram, the gauges continuing to register the tension in the strand. The strand remained in this position for a pre-stressing period of 30 minutes, the load dropping by an amount of about 1,500 lbs. during the first 10 minutes owing to the adjustment of the structure of the strand, and then remaining practically stationary.

At the end of this period the ram was again actuated and the locking pin removed. The tension was reduced in decrements of 10,000 lbs., the movement of the 2,400-foot reference-mark being noted while the strand was held for a short period at each successive tension. This process continued until the tension was all removed, when it was found that there was a permanent lengthening of the strand of about $1\frac{3}{4}$ foot in 2,400 feet. All tensions and extensions were recorded, with the relevant times.

The tension was then increased to the marking load (corresponding to the average dead-load tension of the strand when in place in the bridge structure) of 59,000 lbs., this load being adjusted on each occasion to offset the effect of any difference of the temperature of the strand from the normal temperature of 60° F., for which calculations had been made. A temperature-difference of 5° F. corresponded roughly with a tension of 1,000 lbs. Temperatures were observed on two thermometers (reading to one-tenth of a degree) fastened in contact with the strand, one near each end.

The marks established on the steel plates, as described on p. 418, were then transferred to the strand by set-square and pencil. In order to obviate any inaccuracy due to the difference in tension at the two ends of the strand (arising from friction of the trough), the load was then increased by a further 10,000 lbs. and immediately diminished again to the marking load, temperature corrections being made as before. This had the effect of reversing the direction of the trough friction, the gauge at the north or dead end now showing the higher reading. New marks were then made, and in cases where these did not coincide with the previous marks, a final mark was made halfway between them. If the difference between the marks exceeded $\frac{1}{8}$ inch, the whole marking process was repeated. Permanent paint marks were then made at the various marking points, and, the marking tension still being maintained, a succession of red marks was painted along the top of the strand as a tell-tale in case of any subsequent twisting of the strand.

The whole process of bringing a reel from the shop, unreeling the strand, pre-stressing, marking, re-reeling and returning the full reel generally occupied about 3 hours. Usually, two strands were treated during a night, but, when necessary, three were done.

A personnel of seven men was found sufficient to carry out the pre-stressing, this number being made up of three of the contractor's engineering staff whose duties were to direct operations and make the necessary observations, as well as to establish the strand markings, together with a foreman, mechanic, electrician and carpenter, who between them operated the machinery for pre-stressing, and for reeling and unreeling. The Author was present in a supervisory capacity for the consulting engineers.

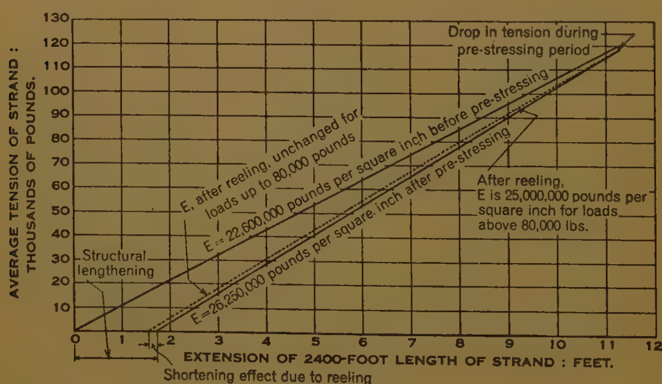
Results of Pre-Stressing.

The results of pre-stressing the cable-strands are indicated graphically in *Fig. 35*, in which a typical stress-strain graph is shown. In accordance with

the specifications, the modulus of elasticity of each of the seventy-four strands was determined during the pre-stressing. Remarkable uniformity was found to exist. The average value of the modulus before pre-stressing was 22,600,000 lbs. per square inch, the value varying between a maximum of 23,000,000 and a minimum of 22,000,000 lbs. per square inch. After pre-stressing, the average value was 26,250,000 lbs. per square inch, with a maximum value of 26,600,000 and a minimum value of 26,000,000 lbs. per square inch.

The footwalk-strands and several of the cable-strands were, as a precautionary measure, reeled up after pre-stressing and then again unreeled and subjected to the marking-load. It was then found that they had shortened by an average amount of 2 inches in 2,400 feet. Further investigation showed that the high modulus of elasticity (26,250,000 lbs. per square inch) which was obtained by pre-stressing, remained unchanged up to a tension of about 80,000 lbs. Between 80,000 lbs. and 120,000 lbs. the modulus fell to a figure of about 25,000,000 lbs. per square inch, during which period the 2 inches of length was regained. As the maximum working load is less than the former figure, the positions of the six marking points were respectively altered by a " reeling correction " proportional to 2 inches in 2,400 feet. As a further check, a few of the strands were unreeled three times. At the second and third unreelings they were subjected only to loads below the critical load of 80,000 lbs. It was found that the length of a strand, after it had been pre-stressed and reeled up again, was unaffected by subsequent reelings. The marking-strands and guide-strands, in order to eliminate any error due to variation of the reeling effect, were actually reeled up and unreeled again before permanent marks were applied to them, the original uncorrected marking-points being used in these cases.

Fig. 35.



Inspection during Pre-Stressing.

Each strand was inspected during the pre-stressing process. Two of the strands were found to have defective galvanizing on one or two of the outer wires. These were returned to the rope-works, where the defective outer wires were removed and replaced by new ones. Pre-stressing was repeated on these two strands, when it was found that the final modulus of elasticity (after pre-stressing) was unaltered in each case. One outer wire of another strand was observed to fail in tension under the pre-stressing load. Examination of the fracture (which was near the middle of the length) indicated " piping " of the

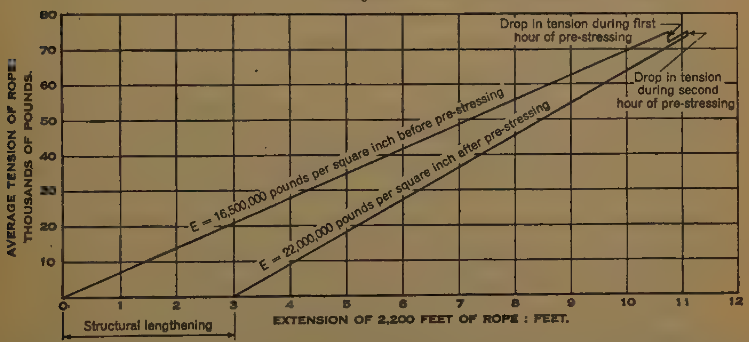
wire, owing apparently to the presence of a minute void in the ingot. This wire was removed and replaced without the strand exhibiting any abnormality during the subsequent pre-stressing.

Suspender-Ropes.

The engineers' specifications required that the suspender-ropes should be manufactured in long lengths, and that each length should be pre-stressed at a load of half the specified ultimate strength, after which it should be measured and marked for cutting while held under a load approximating to the actual dead load in a suspender of the finished structure. The maximum permissible variation of length from the theoretical length was not to exceed $\frac{1}{4}$ inch in any rope.

The ropes were delivered in five long lengths. The pre-stressing was carried out in the same manner as that for the cable-strands. The pre-stressing tension was 75,000 lbs. and it was found necessary to maintain this tension for a period of from two to three hours before the rope ceased to stretch. During this period, the load dropped by about 3,000 lbs. during the first hour owing to the stretching of the rope. It was re-established at 75,000 lbs. tension,

Fig. 36.



after which the further decrease did not exceed about 1,000 lbs. Extension measurements were made on a length of 2,200 feet, a reference-mark established at 10,000 lbs. tension moving a total distance of about 9·75 feet. The permanent stretch in 2,200 feet was found to be approximately 3 feet.

Marking was done at the average dead-load tension of 24,000 lbs. Cutting points were established by hacksaw-marks on the outer wires, and the centre-point of each suspender length was marked with a thin painted line. A series of red marks about 3 feet apart was painted along the top of each rope while it was subjected to the marking-load. These were used in the course of erection to ensure that the suspenders were not twisted, with consequent alteration of length.

A typical stress-strain graph for the pre-stressing of a suspender-rope is shown in *Fig. 36*. The average modulus of elasticity of the ropes before pre-stressing was 16,500,000 lbs. per square inch, the value varying between a maximum of 16,600,000 and a minimum of 16,400,000 lbs. per square inch. After pre-stressing, the modulus of elasticity increased to an average value of 22,000,000 lbs. per square inch, with a maximum value of 22,200,000 and a minimum value of 21,800,000 lbs. per square inch.

The longest suspender-rope being only 236 feet in length, it was not deemed necessary to investigate the effects of reeling upon the suspender-rope lengths.

Discussion.

Professor Lea.

Professor F. C. Lea said that in the first place there was one observation which he wished to make in connection with the floor-slab. The amount of steel used in the floor-slab (which was of very light weight and which was very convenient from the point of view of erection), was far greater than would have been used if an ordinary reinforced-concrete slab had been utilized, but the advantages to be gained from a slab of the particular type employed made that extra steel justifiable. It would be noticed that the tees were welded together, and, fortunately, in the face of the tee where the welds occurred the stress was very small. Assuming that the concrete was stressed to 800 lbs. per square inch, the steel in tension was only stressed to something between 6,000 and 7,000 lbs. per square inch, which rather suggested an uneconomical use of steel. That, however, was quite unimportant in view of the other advantages of the "Teegrid" floor. The welds would be subjected to repeated stresses as loads moved over the structure. He would also like to refer to the welding of the tees to the stringer. A good deal of research had been carried out lately, both in Great Britain (at Sheffield and elsewhere) and in Germany, on the effect of a fillet-weld, or on the discontinuity which occurred when a fillet-weld came on to the flange of a girder. It was found that the repeated stresses per unit area which were possible for a weld of that kind were very much less than the repeated stress which would fracture a plate or a riveted joint; the result was that where the tees were welded to the stringers there was definitely a point of real weakness when considered from the point of view of repeated stresses.

Turning to the wires and wire ropes, the tensile strength of the wire was specified as 220,000 lbs. per square inch for the cables, and the yield-point as 165,000 lbs. per square inch. The Author then stated (p. 371) that the yield-point was defined as being at an elongation of 0·75 per cent. He thought that that must be 0·075, because 0·75 per cent. hardly agreed with what would be expected, and, furthermore, it was rather a high figure from which to define the yield-point.

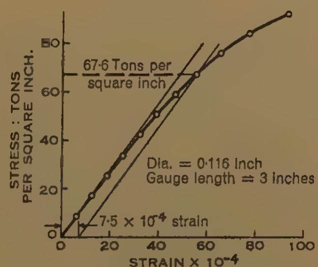
It was apt to be thought that, because wires had a high tensile strength, they would of necessity have a high fatigue-range when subjected to repeated stresses. Recent work had shown, however, that that was quite wrong. The surface condition of a cold-drawn

wire having a strength of 100 tons per square inch was very different **Professor Lea.**
from that of a machined and ground test-specimen of steel of similar strength. The result was that the cold-drawn wire, when subjected to repeated stresses, might have a fatigue-range of less than one-half of what might be expected from a steel of the same strength which had been quenched and tempered, and which had been machined and ground before the test was carried out.

Another point to which he desired to refer was that when the suspender-cables passed over the clamps on the cables there was a definite discontinuity. Further, if it were possible for any part of a suspender-rope to move, by a change in temperature, from a curved to a straight portion of the clamp, a change of curvature resulted. If the resulting stress were calculated it would be found to be very large.

With regard to the tower-saddles, he would like to know if there were a possibility of the cables creeping over the saddles, due to

Fig. 37.



changes in temperature, strain, or load. Such creep would result in a change from a given curvature to a straight line, and he wondered whether it would not be possible to design those saddles with a continuous change of curvature from a straight line to some finite curvature. Any changes of stress due to creep would be superimposed on those resulting from travelling loads, which were indicated in the Paper. It might be argued that creep would only take place at very long intervals; that was true, but nevertheless a considerable number of repetitions of stress might be reached after, say, 60 years.

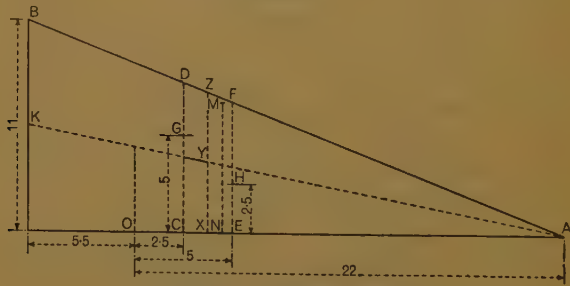
The reason for having to define the yield-point of a steel wire in terms of a particular strain would be clear from *Fig. 37*. He thought it was unfortunate that when a rope was tested and its elongation was measured its modulus of elasticity should be referred to. He suggested that the modulus of elasticity should be considered as a fundamental property of a material and not as something applied

Professor Lea. to a composite structure like a wire rope. Perhaps the term for a rope might be "stress—extension ratio," and "modulus of elasticity" might be kept for the material itself.

Some time ago some of the wires from the Mount Hope and Detroit bridges had been tested in his laboratory. It would be remembered that during the erection of those bridges many wires had been found to be cracked after a considerable amount of work had been done. The cables had had to be taken down, and it had been thought that the wire rope used was defective. Static tests on the wires had revealed no reason for their failure, and the ropes had a reasonable yield-point. It was sometimes argued (wrongly, he thought) that a quenched and tempered wire rope was advantageous on account of the high yield-point thus obtained.

He had carried out a considerable amount of work on the fatigue properties of cold-worked wires. Taking the breaking strength as

Fig. 38.



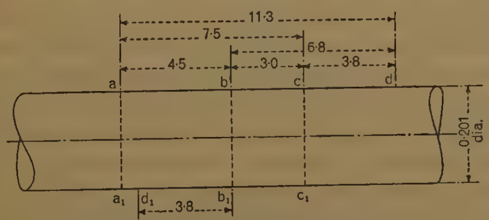
Numbers are to be multiplied by 10,000 to give stress in pounds per square inch.

220,000 lbs. per square inch, the safe fatigue range at zero mean stress would be about 110,000 lbs. per square inch, and the safe range at any specified minimum stress plotted as abscissa would be given by the ordinate of AB (*Fig. 38*).

The maximum stress in the cables due to dead loads was not stated. If it were assumed to be one-third of the maximum stress, then the factor of safety which he suggested was CD/CG $=76{,}000/50{,}000 = 1{\cdot}52$. If the dead-load stress were two-thirds of the maximum stress, then the factor of safety was EF/EH $= 68{,}000/25{,}000 = 2{\cdot}7$; in other words, on those assumptions and assuming a maximum stress in each case of 75,000 lbs. per square inch, the factor of safety was increased by increasing the dead-load stress, or by reducing the range of stress. If it were assumed that the factor of safety was 2 and the maximum stress 75,000 lbs. per square inch, then XZ/XY (*Fig. 38*) would be 2 and OX + XY was to be 75,000 lbs. per square inch. Also $XY = \frac{1}{2}XZ$

$= \frac{1}{2}\left(\frac{220{,}000 - (55{,}000 + OX)}{2}\right) = 41{,}250 - \frac{OX}{4}$. The dead-load Professor Lea. stress was therefore OX = 45,000 lbs. per square inch, and the allowable range for moving loads and temperature changes was 30,000 lbs. per square inch. For a factor of safety 2 and any other specified dead-load stress OX the range was XY. The actual factor of safety would, however, be less than shown by *Fig. 38*, due to concentrated stress, as, for example, at the cable-band castings and at the entrance to the sockets, and due to inequalities of stress in the strand as well as to corrosion and abrasion. The strand-socketing shown in *Figs. 9* did not differ very much from usual practice, but it would be of interest to know whether the properties of the wires were obtained after being heated to 850° F. (the temperature given for the pure zinc) for a short time and cooled at the same rate as that at which the socket cooled. It would be of interest also to

Fig. 39.



Numbers are to be multiplied by 10,000 to give stress in pounds per square inch.

know whether any experiments had been carried out to determine the effect of repeated loads on the zinc fastening.

It would be of particular value to know whether, due to changes of temperature or for any other cause, the rope could creep on to and off the tower-saddles, as, if so, there would be a change of curvature of each wire from straight to a radius of 6 feet $0\frac{5}{8}$ inch. There would thus be a change of stress in the wire of the order $p = \frac{28 \times 10^6 \times 0{\cdot}1005}{72{\cdot}625} = 38{,}000$ lbs. per square inch. The effective modulus of the wire was assumed to be 28×10^6 lbs. per square inch; the radius of the largest wire was 0·1005 inch. That repeated stress could clearly be only infrequently superimposed on the maximum stress conditions. The possibilities were illustrated in *Fig. 39*, in which ad was the maximum possible stress at the top of the wire and bd the range of stress. At the bottom of the wire the maximum stress was a_1c_1 and the maximum range c_1d_1. Assuming ab to be 45,000 lbs. per square inch, the possible range

Professor Lea. was then NM (*Fig. 38*) and the apparent factor of safety was about unity. The number of years required to give, say, 5,000,000 repetitions would probably be very great, however, as such conditions could only occur a few times in one year.

On p. 382 it was stated that the welded pedestal was subjected to a normalizing process, and on p. 383 it was stated that the temperature was 1,150° F. A steel such as would be used in the fabricated welded structure would be a comparatively mild steel, and the upper critical point would be of the order of 900° C., or 1,650° F., so that a temperature of 1,150° F. did not mean a normalizing temperature, as British engineers understood the term. It was really an annealing temperature, and it raised a very important point. It would be ideal if welded structures could be normalized, but most of those who had been working on the problem, however, felt that it was practically impossible to treat even a structure like a saddle to a temperature of anything like 900° C.

Mr. Gill. Mr. E. W. Gill remarked that he had been very interested indeed in what Professor Lea had said about the cables, because one of the first observations that he desired to make about them was that, after the trouble experienced in the failure of single wires on the Detroit-Windsor and Mount Hope bridges, he did not wonder at that type of cable being ruled out. He, too, had wondered if there had been any published results of the examination of those cables, which were found to have been cracked as they went over the saddles on the tops of the towers. With a built-up cable, properly stranded, shop-manufactured and pre-stressed, much more confidence could be felt in its safety. The "locked-wire" construction, as used on the Cologne-Mulheim bridge and as generally used in "Blondin" cableways, would, he thought, have been more favourable than the ordinary stranded cable. Its advantages would appear to be:—(*a*) it would be more compact when assembled in the complete cable, (*b*) it would not require any "blocking out" with cedar packings prior to the application of the final wrapping wire, (*c*) instead of using "thick red-lead paste," a canvas, saturated with red-lead paint, could be wound around the cable, that being done immediately in advance of the wrapping operation, and (*d*) there would, he imagined, be a considerable saving in weight superimposed on the cables by using canvas saturated with red lead and so avoiding the weight of cedar packings and of the fairly considerable amount of red-lead paste used in filling up the voids between the outer strands of the cable, as well as a saving in time and cost. The first cost of the actual cable might, however, be less than for a "locked-wire" construction. Furthermore, a "locked-wire" construction would perhaps be at a disadvantage in offering

less "grip" at the saddles and the suspender cable-bands. Perhaps the Author would comment on those points. Mr. Gill.

On p. 361 the Author said that for wind effects a lateral load (W) of 400 lbs. per linear foot of the bridge was adopted. What wind pressure or velocity was assumed in order to obtain that figure?

The "Teegrid" floor-slab was an ingenious invention, and a saving of 30 lbs. of dead load per square foot was a great advantage. It would, however, be interesting to know to what extent rain-water got through, causing corrosion on the edges of the "T"-flanges in the lengths between the welds. On the top of each "T"-section there was a half-round bar welded to the top of the "T"-iron, which was flush with the concrete of the road. He imagined that the traffic would eventually cause the strips to break off where they had been welded to the "T"-sections. He would have thought that a 2-inch asphalt wearing-surface would tend to prevent water getting through, and would also protect the half-round bars from breaking away.

On p. 372 the Author said: "The strand efficiency, obtained by a comparison of the ultimate breaking load of the strand with the sum of the breaking loads of the individual wires . . . was computed to be 96 per cent." That seemed to be a high ratio. Could it be ascribed to the fact that the wires were wound alternately left and right when making up the strand? The Author stated that the sockets for the cable-strands were filled with nearly-pure zinc, and reference was made to the fact that "the socket was vibrated by hand-hammering, which was continued until the mass ceased to be molten." Although vibration would certainly assist in removing air-bubbles when the metal was in a liquid state, and would assist it in coming into complete contact with the "broomed" end of the strand, was not there a danger of the vibration being continued after the metal had "frozen" but was still hot, thus tending to keep open the boundaries between the newly-formed crystals and preventing the mass forming homogeneously, resulting in it being interlaced with cracks? Had any movement of the sockets been detected during the pre-stressing of the strands? On p. 388 the Author stated: "The suspender-rope, having a total length of 11,547 feet, was manufactured in one continuous operation. . . ." It was also stated that: "Individual wires were spliced when necessary by brazing during the spinning of the rope." Would brazing the wires be satisfactory, as the strength of the brazed joint would not be equal to the strength of the steel wire?

With regard to adjustment, the Author stated that it would be noted that no provision for any adjustment of the suspender-lengths was allowed for, reliance being placed on the accuracy of the shop-

Mr. Gill. work, and (although not mentioned) on the accuracy of the calculated lengths of each suspender. That would appear to have involved unnecessary risks; the least amount of error in the placing of the cable-bands would affect the length of the suspender, while doubtless other small errors would occur, with the result that the stresses developed would differ from those which were calculated. He would like to ask the Author if in a future design he would make provision for some adjustment in the suspender-ropes, such as using turnbuckles. It was stated that each of the four $1\frac{1}{2}$-inch high-tensile steel bolts was tightened up to a specified tension of 33,000 lbs. He would like to ask how that tension was measured, to what degree of hardness the bolts were made, and whether they were of special alloy or heat-treated steel.

The type of expansion-joint evolved to overcome the effects of frost locking together the expansion-fingers was of great interest. He would have thought that, in view of the fact that electric power and light were available on the bridge, it would have been an easy matter to arrange for electric heaters to be placed underneath the expansion-joints. Those heaters could be switched on automatically when the temperature reached freezing point. That would have kept the ice from forming and would have been a very much cheaper arrangement than the one adopted.

He thought that a good deal of unnecessary trouble had been taken in marking each separate strand. Evidently the Author subsequently found that that was so, and, provided that pre-stressing and pre-measurement were accurately carried out and that towers, bents and anchorages were also accurately located, it was only necessary to mark off the first strand; then each strand, as it came across, would be just bedded to its neighbour, and the whole lot built up together.

On p. 410 it was stated that cedar-wood strips had been placed round the cable prior to the cable-wrapping. Those strips were shown as well bedded in Fig. 7, Plate 1; was this obtained by cutting the cedar wood to suit the lay of the wires, or was the power exerted by the cable-wrapping machine sufficient to bed the cedar wood as shown?

Mr. Wilson. Mr. J. S. Wilson said that those who had finally decided to have a suspension bridge in the position described were to be congratulated on their confidence. There were not very many suspension bridges of recent date in the British Empire, and bridges of that type had had a very bad start, both in Great Britain and in North America. The design of suspension bridges had gone ahead to an almost inconceivable extent during the last 110 years. The technique of laying wires and design was now well understood, but in the

early days there had prevailed the most fantastic notions about Mr. Wilson. the design of suspension bridges. Preposterous theories had been put forward, and about 120 years ago an absurd theory of design had been adopted. No engineer or mathematician had been capable of saying that the theory was hopelessly inaccurate, and a considerable number of bridges had been built, many of which had failed under their own weight. There had been a bridge put across the Tees at Stockton which was intended to carry a railway. The bridge was very flexible, and those concerned had not dared to put a locomotive across it; they had put five wagons on, and had pulled them across with a rope.

It was interesting to note that the success of all the modern suspension bridges had depended on the introduction of the stiffening girder, and engineers in Great Britain ought to be proud of the fact that the first suspension bridge with a stiffening girder had been built by a British engineer across the Thames at Marlow. That bridge still existed, and, so far as he was aware, it was the forerunner of all those bridges in the United States which depended entirely on the stiffening girder for their strength. He thought Professor Lea rather exaggerated the high stresses in the wire, and that he was unnecessarily severe on the design. Further, he thought that the possibility of the variation of stress being sufficiently great to reduce the factor of safety through repetition to the extent that Professor Lea had suggested was very remote, and he did not think that the movement in the wires which Professor Lea had suggested was at all likely to occur.

The great point of interest to him in the Paper was the pre-stressing of the strands. To anybody who had had to worry about suspension bridges, and the erection of them, the whole design of a bridge of that type depended on the interaction of the stiffening girder and the suspension system. Both had to deflect, and the real crux of the design was the extent of the deflection which was to be adopted. The deflection depended on the modulus of the rope, and, in spite of what Professor Lea had said, it was quite easy to use the modulus of the rope as long as its meaning was kept clearly in mind. The modulus of the rope was comparatively low as it came from the rope-maker, and pre-stressing (which he had not heard of before in connection with suspension bridges) was, he thought, a very useful advance. When a suspension bridge had to be erected at a distance, it had to be sent in a condition in which it could be put up, and the length of the rope, and the extent to which it would be necessary to screw up the anchorages, all depended on the modulus of the rope. What had been done in Canada by pre-stressing the rope and marking the lengths must have been of considerable value,

Mr. Wilson. and the measurements apparently compared very exactly with the required lengths when the rope was in position.

Mr. Gill had referred to the care with which the sag of the rope had been measured and checked, and mention had also been made of the fact that the hanger ropes had been made without any adjusting screws. It was very difficult to make measurements on a suspension bridge ; it was almost impracticable to measure the length of a rope as it hung between distant towers, and similar difficulty would be experienced in regard to measuring the length of a suspension member. If the suspension members were adjusted the main rope might be pulled out of shape, whereas if it was arranged beforehand for them to be made very accurately in correct lengths, and the positions of the hangers to be marked correctly, all to suit the predetermined camber of the stiffening girder and the sag of the rope, it ought to be possible to obtain as accurate a result as would be obtained by adjustments made at the time of erection. Mr. Wilson thought the evidence to be obtained from the Paper pointed to the fact that, if the strands were pre-stressed and the rope laid up carefully (as had been done in the case under discussion), a good deal of checking and measurement at the time of erection could be dispensed with.

By the pre-stressing the modulus had been raised to about 80 or 90 per cent. of the modulus of the wire. The modulus of elasticity of steel could not be raised, but the raising of the modulus of the strand was due to the drawing together of the wires, bedding one wire into another, and getting proper contact. After having been pre-stressed and carefully measured, those strands had apparently been rolled on drums, taken to the site and unrolled. Did not that rolling upset the lengths and the effect of the pre-stressing ? He imagined that it would slightly upset the lie of the wires, and perhaps would reduce the modulus from the figure that it had been hoped to retain. Another point on which he would like some information was the wind-effect. It was stated that the site was subject to heavy gales, and the bridge was a rather slender structure without guys. The resistance to wind loads was entirely dependent on the stiffness of the platform. It would be interesting to know whether any deflections due to the effect of wind had been measured. A suspension bridge was tremendously strong, but its strength depended on its flexibility, as it had to move to take its load. That was very much against it. The fact remained, however, that in the suspension system steel was used in its strongest sense, and he thought that, without heat-treatment and without rough treatment in the erection of the wire, there was no question that a well-made wire rope of, for example, 90-ton plough steel, was extremely

reliable. If, however, heat-treatment were employed, the reliability of the wire might not be so well assured. Mr. Wilson.

Mr. Gilbert Roberts congratulated the Author on the very complete manner in which he had presented all the relevant facts bearing on the design of the bridge. It was not quite clear in some cases, however, whether the loads referred to were for the full width of the bridge or for one truss only. Mr. Roberts.

Turning to the live loads specified, he thought it was rather striking that such a small load per square foot had been used in the design of a bridge of the kind under discussion; he would like to ask the Author whether any tests had been made to determine the concentration of live load likely to be met with on the bridge, or whether the loads had been taken to cover traffic on to an island as distinct from a main road. It would be seen that the normal live load was 900 lbs. per linear foot of bridge, and that the so-called congested live load was 1,200 lbs. per linear foot. Those figures corresponded to 45 and 60 lbs. per square foot of roadway respectively.

Fig. 40.

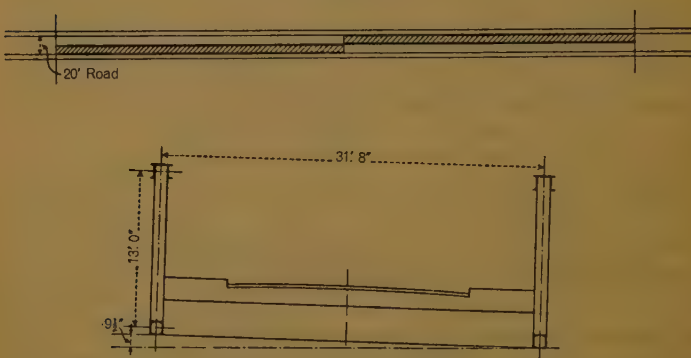
The load per square foot that was required by the Ministry of Transport in Great Britain for the design of such bridges was about 200 lbs., which was between three and four times that which had been used in the design of the bridge described in the Paper.

As Mr. Wilson had said, the characteristic of suspension bridges was the deflection of the cable and of the stiffening truss which was attached to it. That deflection, or distortion, might occur independently of the extension of the cable; supposing that the cable remained of the same length, and that a part of the bridge were loaded, a local deflection might still be obtained, due to the distortion or contortion of the cable. That might be of little importance if both the cables and both the trusses deflected equally, because the change in grade at the end of a suspension bridge of the kind under discussion, due to a maximum deflection, was only of the order of 1 in 100, which was hardly noticeable to traffic; it might, however, become important if loads on one side of the roadway were going to be considered, resulting in a twist on the deck. He had made some calculations of those deflections, using the data as accurately as he could take them from the Paper, to indicate what those deflections might be. *Fig. 40* showed the deflection at the

Mr. Roberts. two quarter-points due to the normal live load of 900 lbs. per linear foot on the full width of the roadway, each cable taking half the total load of the roadway. The congested load would produce deflections one-third greater than those. If opposite corners of the bridge were loaded (*Figs. 41*) the difference in deflection between the two trusses was 0·31 of the sum of the two deflections, or about 9½ inches, or 12½ inches for the congested live load.

At first sight those figures might look rather alarming, as a cross slope on the roadway of about 1 in 30 would be distinctly noticeable; it had, however, to be considered whether that was a condition of loading which was likely to occur. The only way to

Figs. 41



reduce those deflections in suspension bridges was either to increase the dead load and so the resistance of the cable to distortion, or to increase the moment of inertia of the stiffening truss. Both those expedients were expensive, and he thought it was very doubtful whether the additional expenditure would be justified in order to cater for a condition of loading such as that which he had described, and which was improbable. He noticed that the Author said that discontinuous loads had been used in the design of the bridge, and he would like to ask whether the Author had considered that problem of the twisting of the bridge and the relative deflections that were set up between the trusses.

Mr. Gribble. Mr. Conrad Gribble remarked that there were one or two points in the Paper to which he would like to refer. It was not the first case in which cables had been pre-stressed; he had seen in a report [1] on the high-modulus foot-bridge ropes for the Fort-Lee—Hudson-River bridge that the cables for that bridge had been pre-stressed

[1] *Engineering Abstracts*, **45**, 123, Inst. C.E., 1930.

Mr. Gribble. in a very similar manner to that adopted by the engineers for the Island of Orleans bridge, and there was one very remarkable agreement between the result obtained in the Fort-Lee—Hudson-River bridge and that obtained in the Canadian bridge. It was stated at the end of that short report[1] that "With proper fabricating machinery and pre-stressing, a modulus of elasticity of 88 per cent. of the wire modulus can be obtained." Assuming that the modulus of the wire used in the Island of Orleans bridge was 30,000,000 lbs. per square inch, the maximum figure of 26,600,000 lbs. per square inch for the rope was almost exactly 88 per cent. of the former figure, and it might be assumed that that was probably the result that could be obtained with wire ropes. The ropes in the former bridge were considerably longer, being 3,750 feet in length, but the same results had been obtained.

In the suspension bridges which the Author mentioned the ratio of sag to span was nearly always about 1 to 10. The Author had not given any reason why that sag/span ratio had been adopted. He did not suppose that the Author had merely taken that ratio because it had been adopted in other bridges, without having made some check of its suitability from the standpoint of economy, and if he could enlighten him in that respect it would certainly be of some interest.

Mr. Webster. Mr. T. H. Webster said he had been very interested in Professor Lea's theory regarding the possibility of relative movement between the cable and the saddle, but would not the saddle contract at the same rate as the rope would contract, it being subject to the same variations in temperature? If so, there would be no relative movement and therefore no change in curvature.

Mr. Gough. Mr. G. S. Gough remarked that two points had occurred to him on reading the Paper. In the first place, there was a footnote on p. 365 in which the Author suggested that the web system of the stiffening girders contributed to the stiffness of the girders. It seemed to Mr. Gough that that was entirely erroneous; the web system could only introduce shearing deflections, and actually reduced the stiffness.

On p. 379 there was a formula for the stressing of the towers. He found that formula rather curious, because it gave a minimum allowance for the bending at the top of the tower, and it also gave the same at the bottom. Surely the towers had their maximum bending moment at the bottom.

With regard to the possibility of repetition of stress, about which Professor Lea had spoken, if there were important temperature

[1] *Loc. cit.*

Mr. Gough. effects how often did they occur ? Surely in Canada the temperature would only be near some critical value during part of the year, and if it changed three times a day the number of damaging reversals could hardly exceed a thousand in a year. Taking, then, 100,000 as the limiting number of reversals, it seemed that the bridge could last 100 years without the repetition of stress seriously affecting the factor of safety.

Mr. Hamilton. Mr. D. M. HAMILTON said that he had visited the Island of Orleans bridge in February, 1935, and he would like to clear up, if possible, some of the points which had been raised. With regard to the question which had been put regarding slip in the sockets of the cables, which were sunk at the ends in zinc, as far as he knew no slip of the cables or of the strands themselves in the sockets had been experienced. Professor Lea had spoken of normalizing at 1,150° F. That seemed to be the common temperature for "stress-relieving" (as it was called in America) in large welded components. The Dominion Bridge Company had manufactured the saddles and the bases, and they had based some of their work on the results of research work at McGill University, at which it had been found that stress-relieving at a temperature of 1,150° F. on small test-pieces had reduced the residual stress in the weld, and in the metal adjacent to the weld, down to about 5,000 lbs. per square inch. That was a considerable reduction from the stress that occurred in welded pieces that had not been stress-relieved, which amounted in some cases to 30,000 lbs. per square inch. While he would not go into the question as to whether there was any change in crystal structure, it had certainly been shown in some cases that normalizing, or stress-relieving, at 1,150° F. had had a great effect in the reduction of residual stress.

With regard to the suggestion that heaters might have been installed at the expansion joints in the bridge, it was quite possible for the temperature to fall to —50° F.; he did not, however, know how much current the heaters would use when the temperature fell to that figure. The same speaker had dealt with the trouble which might occur when the road surface had been sufficiently worn down for the tires of the vehicles to encounter the steel of the flooring. He understood that the traffic on the bridge was very small, and that probably by the time the traffic had worn the roadway down to the steel other parts of the bridge would be in a bad state of repair as well.

Correspondence.

Mr. D. B. ARMSTRONG, of Lachine, Quebec, observed that he had been closely associated with all phases of the work in the capacity of Supervising Engineer for the contractors. The matter of design loads and allowable stresses was always of considerable interest. The loading which governed the cable design was a "congested load" of 1,200 lbs. per linear foot over the entire length of the bridge, equivalent to 60 lbs. per square foot of roadway, or a total load of approximately 1,140 tons, which was assumed to act in conjunction with a temperature of —20° F. The minimum "defined" yield point for the individual cable-strands was 167,000 lbs. and in tests was found to average 170,000 lbs., which was equivalent to 150,000 lbs. per square inch. Whilst it was recognized that the influence of temperature was relatively small, the maximum allowable stress of 75,000 lbs. per square inch would appear to be extremely conservative for the assumed congested loading, which was unlikely ever to occur, and would certainly not occur at such a low temperature. Mr. Armstrong.

In designing the tower columns the stresses due to axial loads, the longitudinal deflection of their tops, and the flexure caused by the eccentricity of the loads had all been accounted for. Was it necessary, therefore, to apply a formula to restrict the primary stresses in the body of the columns, or was that done to provide against such secondary influences as wind, local distortions, and errors of fabrication?

The incorporation of a spherical bearing surface in the anchorage and the provision of threaded connection-bolts proved to be a great advantage in many ways. That arrangement made it possible to fabricate all the strands to the same length, since the minor theoretical differences ($2\frac{1}{4}$ inches maximum) from the mean length could be taken up in the bolt adjustments in the field. Therefore, with the exception of the specially marked guide- and suspender-location strands, the strands required no special identification other than marks denoting whether they were of right- or left-hand lay; that was a great advantage in both manufacture and erection. The use of eyebolts also obviated the necessity for the customary adjustment-shims, and provided a simpler, quicker and more accurate method for making the final adjustments. An important

Mr. Armstrong. advantage of the spherical bearing was that all strands were radially connected, and the bending at the socket-faces, which was an objectionable feature common to most types of anchorages, was entirely eliminated.

There were certain additional points which might have been mentioned in the Paper. Provision had been made for shimming between the rocker-posts and tower-struts (Figs. 5, Plate 1) to compensate for discrepancies between the theoretical and actual elevations of the ends of the stiffening-trusses after the full dead load had been placed; such shimming was, however, found to be unnecessary. The elevations of the two cables at the centre of the main span were found to be within $\frac{5}{8}$ inch of each other, and within $\frac{3}{4}$ inch of their theoretical elevation when the final survey was made. The deflection of the main span under a central test load of 42 tons amounted to $6\frac{7}{8}$ inches.

Further, a striking demonstration of the close cohesion between the several layers of wires in the cable-strands had been made when testing one of the 100-inch specimens to destruction. Internal breaks occurred at three intermediate stages of loading before the strand finally failed under a tension of 252,000 lbs. On inspection, it had been found that the three preliminary breaks had all taken place in one wire (which was defective, due to piping), showing conclusively that a very positive bond existed between the strand-wires; that fact largely explained why the high efficiency of 96 per cent. was possible for the strand as a whole.

In connection with the erection, the contractors had been required by specification to prepare their programme and to submit it to the engineers for approval. Having had no previous experience with that type of erection, the contractors consulted Messrs. Robinson and Steinman, consulting engineers, of New York, whose excellent advice and generous assistance was of great value to them in that phase of the work. Amongst the suggestions made by them was the simplified method of equating the cable-strand and span lengths (p. 401). Mr. Armstrong understood that that was developed by Mr. J. London, a member of their staff, who was also responsible for the convenient form of chart for the cable-strand adjustments (p. 403). The cable-wrapping machine (*Fig. 31*) was an improved type of that originated by Dr. Robinson some years ago, and was built from plans prepared in their office.

The cables were erected under extremely adverse weather-conditions in sub-zero temperatures, and at a time when the site was almost incessantly swept by strong winds. Wind-breaks were constructed on the towers and at other strategic points, despite which the workmen were frequently frost-bitten and had to suspend

Mr. Armstrong.

work for that reason. It was twice found necessary to readjust the strands after strong gales had disarranged their setting, in spite of the numerous clamps used to guard against such an eventuality. On one occasion a thaw had set in, accompanied by rain, and when colder weather returned two days later, the strands and footwalks were left sheathed in ice. The latter was removed from the strands as far as possible by beating and scraping them with hardwood sticks, metal scrapers being avoided for fear of damaging the galvanizing. Since many of the galvanized strands remained to be drawn across the footwalks, no sand or chemical could be applied to the latter to improve the footing, and for many days, until the ice had evaporated, walking on the footwalks was, of necessity, uncertain and hazardous.

He subscribed to the Author's opinion that it was entirely feasible to dispense with most of the elaborate guide-strand adjustments in the field, but with the proviso that to do so it was essential that all preliminary work should be carried out with extreme accuracy. In that instance especial care was exercised in the field surveying, in fabricating and erecting the towers, and in pre-stressing and marking the strands, where the greatest refinement of stress measurement and control was essential to obtain accurate results.

In the absence of suitable triangulation-stations, the river-crossing was measured in the winter by means of a long piano wire which was suspended at suitable intervals from trestles set up on the ice. That wire was set up later in an identical manner on the land, and its length, corrected for temperature, accurately established. The base line of the pre-stressing plant was chained with the same tape and by the same engineers. Before any guide-strands were measured or marked, an extensive study had been made to determine the full effects of pull, temperature, friction, and reeling on the final length of the strands, and it was believed that the total error from all those causes was ultimately limited to less than $\frac{1}{2}$ inch.

Every care was taken in the shop-work on the towers. For the facing of the joints, the sections were aligned in the milling machine by means of a transit. Adjoining sections were afterwards assembled together and brought to full bearing at the joints by means of turnbuckles. They were supported on screw jacks and their alignment was checked by means of a surveyor's level and a piano wire stretched between the two extreme ends, before the splice holes were reamed to the full size. All splice material was match-marked. In dressing the tops of the main piers to receive the tower-shoes, a very sensitive level (containing the bubble from a surveyor's level) was used in combination with a long steel straight-edge, to ensure a perfectly

Mr. Armstrong. horizontal surface. During the erection of the towers, the outside corners of the splices tended to remain open, due to the batter, the camber and the unbalanced creeper-traveller loads, but that trouble was rectified by temporarily substituting special connecting plates for some of the permanent ones. Those plates had groups of holes corresponding to the splice holes, but spaced slightly closer together than in the members to be joined. By means of drift-pins the joints were forced to a true bearing and held there while the surrounding rivets were driven, after which the temporary plates were replaced by the permanent ones.

It was by the observance of the aforementioned and many similar precautions that the construction errors were held within such close limits, and in the final analysis it was felt that the results obtained fully justified the extreme care that was exercised throughout all stages of the work.

Mr. Kennedy. Mr. DUNCAN KENNEDY observed that, from the description given in the Paper of the erection of the main cables, it appeared that one guide-strand on each side was placed in position and adjusted, and thereafter the remaining strands were placed in the saddles and adjusted in accordance with the position of the guide-strands. He would like to ask the Author whether any trouble had been experienced owing to inelastic extension taking place after the strands were erected. On p. 405 it was stated that the guide-strands were in place on 14 December, 1934, but final adjustment observations were not made until 3 January, 1935, so that presumably about 20 days elapsed between the erection of the first and second strands of each cable. If any stretching took place due to the time-factor, a corresponding allowance would appear to be necessary when the other newly-placed strands were adjusted to the sag of the guide-strands.

Mr. Legget. Mr. R. F. LEGGET, of Montreal, considered that there were several features of the project which seemed to merit more attention than had been given to them in the Paper. The economic aspect of the bridge-design might, for example, have been discussed. He had seen and studied the bridge, and he could not subscribe to the æsthetic fitness of the design adopted; he would, therefore, like to see some details of its economic justification. Æsthetics were always a debatable topic, but surely the assertion on p. 378 that "the strict austerity of the design was softened by an arching of the top strut and of the diagonal members immediately above the roadway" was more than questionable, the striking functional design of the towers being weakened as well as softened by that modification.

With regard to design, would the Author explain why another column formula (p. 379) had had to be evolved, in view of the number

Mr. Legget.

of such formulas already published? The special floor-system introduced into the bridge structure was of interest, and information regarding the holders of the respective patents would be of use. Could the Author give further particulars of the tests made on the "Teegrid" slab, since it would add to the value of the data he had already given if the nature of the failure were known? The use of the model of the anchorage pier, referred to on p. 394, was an unusual feature of construction well worthy of elaboration, since that apparently simple device was all too often neglected on foundation work of the nature called for by the bridge described in the Paper. Although repeated reference was made to the "guide-strand," no explanation of that term was given: was it correct to assume that it was one of the main strands used as a guide? Some data on the paint used, in addition to its colour, would be useful in view of the exposed location of the site.

The Author had made no reference to the prominent part played in the construction of the bridge by the Dominion Bridge Company. In particular, the notable advances in welding technique developed by the firm for the bridge might have been mentioned.[1] Public descriptions of the work given in Canada by Mr. P. L. Pratley had included due appreciation of the part played by the contractors, and it was therefore to be hoped that knowledge of that co-operation would not be confined to Canada.

Mr. McConnel.

Mr. Samuel McConnel, of Nairobi, mentioned that the theory underlying the design of suspension bridges had been ably dealt with by eminent German and American engineers, and the use of the deflection theory had enabled considerable savings to be effected, especially when a comparatively flexible stiffening girder could be used; the large number of structures of that type built during the last few years had enabled designers to obtain a sound knowledge of their economic design.

Efforts had constantly been made to reduce the weight of the decking, and the design used for the Island of Orleans bridge, which was of a similar type to that employed in several other bridges in the U.S.A., was a decided advance. Many suspension bridges in the past had been decked with timber, while recent French practice had favoured the use of a reinforced-concrete slab. For the proposed reconstruction of the Brooklyn bridge Dr. D. B. Steinman had suggested[2] the use of a 1½-inch asphalt surfacing supported by a decking of aluminium alloy 27 S.T., consisting of 9-inch channels at

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[1] D. Boyd and G. Cape, "Distortion Control Procedure." Iron and Steel Institute, Symposium on the Welding of Iron and Steel, vol. ii. (1935), p. 39.

[2] "Aluminium Trusses and Floor for Brooklyn Bridge," *Engineering News-Record*, vol. 114 (1935), p. 547.

Mr. McConnel. 8-inch centres covered by a plate $\frac{7}{16}$ inch thick, which would only weigh 32·5 lbs. per square foot, although designed for a much heavier concentrated loading than that employed in the present bridge. Among modern suspension bridges there had recently been erected across the Durance at Cavaillon (France) a span of 997 feet with very handsome concrete pylons, whilst a self-anchored bridge of 856 feet span had been put up at Belgrade. Although American authorities had stated that the economic depth of the stiffening girders was about $\frac{1}{45}$ to $\frac{1}{50}$ of the span, that view was not shared by French engineers, who employed ratios as low as $\frac{1}{100}$ of the span for highway bridges, whilst in the case of spans up to 300 feet they utilized broad flanged beams about 39 inches deep for structures designed for two lines of traffic. Although wire cables were almost invariably used, the strands were kept some little distance apart so that individual cables might be replaced if required. Plate girders which also served as parapets were nearly always used for stiffening girders, and in recent examples they had been fabricated of chromium steel. The towers were normally hinged at the base. In recent bridges stresses as high as 88,000 lbs. per square inch had been specified for the cables, whilst in modern practice the cables were allowed to be stressed up to 60 per cent. of the elastic limit.

It was of considerable interest to compare the weights of the Island of Orleans bridge and of that at Florianopolis, Brazil. The latter bridge had eyebar chains of special heat-treated steel, which were naturally much heavier than wire cables, and a timber deck. In addition, the live load on the Florianopolis bridge was taken at 2,200 lbs. per linear foot including impact, as compared with 900 lbs. per linear foot in the Canadian bridge, as the former bridge was designed to carry a light electric railway. Comparative details were as follows :—

Bridge.	Total length: feet.	Effective width: feet.	Weight: short tons.
Florianopolis	1,857	37	3,080
Island of Orleans	1,775	30	2,634 *

* Weight of 236-foot side spans and floor grid omitted.

The Florianopolis bridge involved a most unusual form of stiffening truss utilizing part of the cable, but a bridge of 600 feet span at present under construction near Brisbane (Queensland) appeared, from accounts in the lay Press, to be of the same type except that wire cables were used in lieu of eyebars.

For light traffic many bridges of considerable span existed in

which stiffening girders had not been employed; notable examples of that type were those at Rio Chiriqui (Panama) and at Royal Gorge (Colorado). He hoped that the Author would supplement his Paper with details of the relative deflections of various parts of the bridge, and that he would express his views as to the degree of rigidity necessary for highway bridges. Mr. McConnel.

Mr. C. D. Meals, of Hamilton, Ontario, observed that, in the United States, it had been the wire-rope manufacturer who had done all the work on the cable-strands and suspender ropes, including the pre-stressing, measuring, and attaching of sockets; that practice had been fostered by them, principally on the basis that the steel-fabricators were hardly qualified to do the work. It was a credit to the Dominion Bridge Company that the supposed difficulties were not so great as anticipated, and that they had accomplished the cable-strand and suspender-rope work with only a few minor difficulties, which were so ably explained by the Author. Practically no information was given in the wire-rope manufacturers' catalogues relative to the physical properties of cable-strands and suspender-ropes; that applied equally as well to the British, Canadian and American manufacturers, with the exception of the John A. Roebling's Sons Company, of Trenton, N.J., and the B. Greening Wire Company, of Hamilton, Canada. Mr. Meals.

Appreciating the lack of such data, he had recently written an article[1] in which very full data were given regarding cable-strands and suspender-ropes. The physical properties of those strands and ropes were as follows, in terms of the strand or rope diameter "d":

PHYSICAL PROPERTIES OF STRANDS AND ROPES.

	Cable-strands.	Suspender-ropes.
Breaking strengths: short tons . . .	$62d^2$	$44d^2$
Weight per linear foot: lbs.	$2{\cdot}08d^2$	$1{\cdot}70d^2$
Metallic area: square inches . . .	$0{\cdot}590d^2$	$0{\cdot}460d^2$

In that article, formulas were given for the calculation of the breaking strengths of the cable-strands and suspender ropes, from which the efficiencies might be determined, and it might be of interest to note the actual test and calculated efficiencies for those members as used on the Island of Orleans bridge, which were as shown in the Table on the following page.

[1] C. D. Meals, "Main Cables and Suspenders for Suspension Bridges." Journal Eng. Inst. Canada, vol. xvii (1934), p. 358.

Mr. Meals.

TENSILE STRENGTH EFFICIENCIES OF STRANDS AND ROPES.

	Efficiency: per cent.	
	Tests.	Calculations.
$1\frac{3}{8}$-inch, 1 × 37 cable-strands . . .	96	94
$1\frac{3}{8}$-inch, 6 × 19 I.W.R.C. suspender-ropes	91	89·2

It would be seen that there was a close agreement between the test results and the calculated values.

It was noted that both the rope and the independent wire-rope centre of the suspender-ropes had been made right-hand regular lay "in accordance with common practice." The latter remark was, however, incorrect, as, for example, the suspender-ropes for the Portsmouth, Mount Hope, Philadelphia-Camden and Detroit-Windsor bridges were of right-hand regular lay, but the independent wire-rope centres were made right-hand Lang-lay, and from an engineering basis the latter practice was essentially more correct.

The suspender-rope specification requirement of a certain strength of rope per part, as bent over a sheave, had been given considerable thought in the United States in the past few years; three articles [1, 2, 3] had been published with reference to that question, but further testing would be required before something definite was known relative to the subject.

Mr. Meals had presented a formula [4] to determine the breaking strength of a wire rope bent over a sheave under static loading, in which formula the modulus of elasticity of the rope (as measured during the first application of pre-stressing load) was a factor; recent investigations,[2, 3] and the tests of the $1\frac{3}{8}$-inch suspender-ropes, however, seemed to indicate that the rope modulus was not necessarily a factor, but the proof was not conclusive. If the rope modulus could be ignored it would be a desirable feature in suspender-ropes, as it would permit the usage of ropes with a high modulus of elasticity with a resultant decrease in their stretch and a resulting decrease in the stresses of the chord members of the truss. In some preliminary calculations made for the $1\frac{3}{8}$-inch ropes in January, 1933, he calculated the efficiency, as bent over the 14-inch sheave,

[1] E. Skillman, "Some Tests of Steel-Wire Rope on Sheaves." U.S. Bureau of Standards Technologic Paper No. 229, 1923.

[2] F. C. Carstarphen, "Effects of Bending Wire Rope." Trans. Am. Soc. C.E., vol. 98 (1933), p. 562.

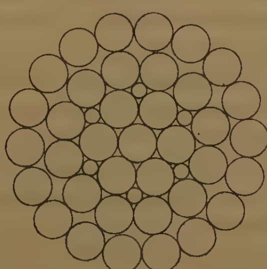
[3] D. M. Stewart, "The Behaviour of Stationary Wire Ropes in Tension and Bending." Proc. Am. Soc. C.E., vol. 62 (1936), p. 161.

[4] *Loc. cit.*

as 78 per cent.; actually, it was 82 per cent. The 1×37 cable-strand shown in Fig. 8, Plate 1, was not geometrically correct as, actually, the strands were made as shown in *Fig. 42*, the difference being the relation of the twelve wires to the six large and six small filler-wires, which they enveloped. Mr. Meals.

Reference had been made to the attaching of the sockets on the ends of the cable-strands and suspender-ropes, and to the fact that a commercially pure zinc or spelter had been used; the usage of zinc was at a variance with the British practice of using a white-metal, which was in accordance with the recent B.E.S.A. Specification for Capping Metal for Steel Wire Ropes (B.S.S. 643); British practice was, however, at variance with both Canadian and American practice in that respect as, in the latter countries, zinc only was used

Fig. 42.



by the wire-rope manufacturers; he had noted that fact in a recent article.[1]

The Author had been modest in his reference to the Island of Orleans bridge being Canadian in design and fabrication; the Canadian-made cable-strands and suspender-ropes compared most favourably with similar products made by the leading American wire-rope manufacturers, who had had a much more extended experience.

Mr. L. S. Moisseiff, of New York, considered that modern traffic made several specific demands on the strength and capacity of roadways for highway bridges. One of those demands was the provision for higher load concentrations and therefore greater strength of the floor-system of a bridge. That was due to the heavier axle-loads on freight-carrying trucks, and another demand was for a continuous non-skidding roadway surface which would change little with variations in temperature and humidity. It was also desirable that Mr. Moisseiff

[1] C. D. Meals, "British *v.* American Wire Rope Practices." *The Wire Industry*, vol. 2 (Sept. 1935), p. 315.

Mr. Moisseiff. the roadway floor should be fireproof. Those requirements added to the weight of the floor system and roadway. On the other hand, the load from truck and passenger traffic was, on the average, not high, and, moreover, it was fairly uniformly distributed. The live load per linear foot of lane could be taken as fairly moderate for spans of some length. The utilization of the suspension-bridge type as it had been developed in more recent times made it possible to build relatively long spans very economically. That was well demonstrated by the successful execution of the Island of Orleans suspension bridge.

The designers of the bridge were well informed on the advances realized more recently in suspension bridges, and they had made full use of their knowledge. The Paper showed that the latest achievements, both in theory and practice, had been applied in the design and construction of the bridge. The use of a steel-and-concrete grid floor for the roadway was one of the examples of a recent advance in bridge engineering.

Having in view the development of an efficient and fireproof floor for highway bridges of longer spans which would be light enough to realize economy in the cost of the bridge, he had developed the reinforced-concrete slab type which had been used for the roadway of the Delaware River bridge at Philadelphia. That type was based on the embedding in concrete of shop-fabricated steel reinforcing trusses, usually spaced about 6 inches apart. The provision in the design of those trusses of diagonals or webs definitely capable of taking the shear in the slab, and the definite dimensions and placing of the trusses, had made it possible, in that case, to construct a slab with a total thickness of not more than 6 inches. The embedded steel reinforcing trusses were $4\frac{1}{2}$ inches deep, leaving a full concrete cover of $\frac{3}{4}$ inch at the top and bottom. On that bridge a 2-inch wearing surface of asphalt had been placed on top of the concrete slab. Subsequently, tests were conducted on two large slabs of that design, measuring 19 feet by 19 feet, by the Delaware River Bridge Joint Commission jointly with the United States Bureau of Public Roads. Those tests had shown the great efficiency in distribution and strength of that type of slab, and had furnished much interesting information on slab behaviour.[1] With the evolution of modern concrete roads and the extended experience with them, it had become, since then, general practice in the United States to add to the required design depth of the concrete slab a wearing-surface of from $\frac{3}{4}$ inch to 1 inch in thickness. That wearing-

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[1] G. W. Davis, "Tests of the Delaware River Bridge Floor Slabs." *Public Roads*, vol. 8 (1927), p. 159.

surface formed a part of the slab and was made simultaneously with it. Under certain conditions of load and spacing of supports the roadway slab might easily be made an inch thinner. Mr. Moisseiff.

A 7-inch slab of concrete reinforced by steel trusses would weigh about 92 lbs. per square foot, but vibrated concrete would weigh about 155 lbs. per cubic foot, so that a vibrated slab would weigh about 100 lbs. per square foot. Under special conditions, a light-weight concrete aggregate might be used, which would reduce the weight of the slab ; such a slab had been used for the upper roadway of the San-Francisco—Oakland-Bay bridge at present under construction. That roadway was only constructed to carry passenger motor traffic, enabling lower axle-weights to be used. A 6-inch slab, including a 1-inch wearing-surface, was built for it, with a weight of 62 lbs. per square foot.

He had, in 1928, advised and made large-scale tests [1] of slabs of the first steel-and-concrete grid which had been developed. Those tests proved the good qualities of that type of construction, and it had been installed on a number of bridges. The " Teegrid " slab used in the Island of Orleans suspension bridge was of the same type and construction, except that the upper cross-bars were welded while in the former type they were gripped. As pointed out by the Author, the weight of that slab was 50 lbs. per square foot, and he had effected thereby a saving of 30 lbs. per square foot or 600 lbs. per linear foot of bridge. The armoured surface of the roadway which was obtained by the steel-and-concrete grid had been tested and rated very highly as to resistance to skidding, while at the same time the wearing resistance of the slab ought to be greatly enhanced.

Another advance made by the engineers of the Island of Orleans bridge was the replacing of the tower and cable-bent saddles by steel slabs welded to the proper form, thus avoiding the difficulty of obtaining large steel castings which were free from faults.

He was able to obtain large forgings, costing less than steel castings, for the pin bearings of the Bayonne (Kill van Kull) arch. Steel forgings for suspension-bridge saddles were, however, more difficult to manufacture, and much more costly.

Mr. John Portas, of New Glasgow, Nova Scotia, observed that there could be little argument regarding the æsthetic superiority of the suspension bridge, and he had been convinced by work he had done on alternative projects, particularly on a project for a bridge over Halifax harbour, of the economy of the suspension type for highway bridges of medium and long span, where the foundation Mr. Portas.

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[1] L. S. Moisseiff, " Development Tests on a Light Floor for Bridges." *Engineering News-Record*, vol. 104 (1930), p. 71.

Mr. Portas. conditions were at all favourable. Alternative bids had generally shown that, for spans up to 1,200 or 1,500 feet, the pre-stressed rope-strand cable had advantages in economy and speed of erection over the parallel-wire type.

He considered that the loadings for the Island of Orleans bridge were normal and the stress-combinations reasonable; the concentrated loading, consisting of two 15-ton trucks abreast, was entirely adequate, although much lighter than the very heavy concentrations which placed such a burden on long-span bridge-construction in England. The use of lightweight floors such as the "Teegrid" design which had been developed recently in the U.S.A., had advantages due to the reduction in dead load and to the increased speed of construction, but he did not, however, agree with the Author that a decrease in the weight of the floor would result in a reduction in the weight of the stiffening truss; in fact, he suggested the contrary to be the case. Under dead load and at normal temperature the stiffening truss was unstressed. By the "exact" method of stress-determination, moments due to live load and temperature depended not only on those loadings but were also functions of the dead load; the effect of the dead load was to reduce the live-load and temperature moments and shears for which the stiffening truss was designed. The shallow stiffening truss, with a depth of only 13 feet, or $\frac{1}{81}$ of the span, was in agreement with modern practice, as it was recognized that the resistance offered by the truss to cable deflections was very small, and that its function, so far as vertical forces were concerned, was to prevent local distortions. Since the unit stresses in the truss depended principally on cable deflections they were not greatly influenced by the areas of the truss members.

It would be of interest to know what relation the preliminary stiffening truss, as designed by the "elastic" method, bore to that finally arrived at by the "exact" method. Some years ago Mr. Portas, under the direction of Mr. P. L. Pratley, had investigated the design of the stiffening truss of Detroit suspension bridge. The preliminary design, using the "elastic" method of analysis and assuming a truss depth of 36 feet, had resulted in a truss with an average moment of inertia of approximately 124,000 inch^2-feet^2 units. Later, the depth had been reduced to 22 feet and the moment of inertia to 44,000 inch^2-feet^2 units, and final calculations by the "exact" method had resulted in the use of a truss with a moment of inertia of about 18,000 inch^2-feet^2 units. The weight of the truss as designed by the "elastic" method had been found to exceed by about 52 per cent. that by the "exact" method. Preliminary investigation of the system by the approximate method

was helpful, and influence-lines so constructed might be used with advantage to determine approximately the points of maximum moment and shear, but, in view of the above figures, Mr. Portas suggested that a closer approximation of truss sections could be based on judgment. Figures giving the maximum truss-deflections would have been of interest, and some explanation of Mr. Pratley's formula, used in the design of the towers, would increase its value to the engineering profession. Mr. Portas.

With regard to the stresses in the trusses due to wind load, the Author apparently meant that the top and bottom chords participated in proportion to their areas (not equally), the induced hanger-loads being such as to equalize the strain in the chords. Important bridges, with stiffening trusses of the " pony " type, had been built with no provision for that interesting stress distribution.

Was the Author's reference to Messrs. Dorman, Long's " Chromador " steel as silicon steel correct ? Structural silicon steel as standardized in American practice was not an alloy steel in the generally-accepted meaning of the term, but was a high-quality medium-carbon steel, with a silicon content of from 0·20 to 0·35 per cent., only very slightly above the normal percentage of residual silicon that was found in steel. The silicon produced no change in the structural constituents of the steel and its strengthening effect was thought to be due principally to its de-oxidizing qualities. " Chromador " steel, on the other hand, was a chromium-copper alloy, with a specified silicon content not exceeding 0·2 per cent. Chromium was a true alloying element which modified the grain structure and tended to diffuse the pearlite areas, and by the action of the alloyed elements in the ferrite it gave a steel which combined high strength with workability. That result was obtained without the heat-treatment which was usually considered necessary to bring out the value of chromium as an alloying element.

New features introduced in the Island of Orleans bridge were spherically-machined slabs at the anchorages, ladder-rungs on the suspenders, and the link device to prevent freezing up of the expansion joints. It was to be hoped that the last feature would prove to be a satisfactory solution of a minor but very real problem in Canada. The wire-mesh fence, which had proved satisfactory on other light suspension bridges, was adequate considering the fact that the stiffening truss was not part of the primary structure. A curb at least 9 inches high, as called for in the Specifications of the Canadian Engineering Standards Association, might, however, have been provided. The hoisting of the stiffening-truss sections from the ice on tackles attached to the main cables was an erection-process peculiar to bridge-construction in Canada, and was first

Mr. Portas. used on the Grand' Mère suspension bridge. The behaviour of ice 2 feet thick under loads of 15 tons was of interest.

The detailed description of the pre-stressing of the cables was a valuable contribution to engineering literature. He had been present one night during the pre-stressing operation at Longueuil, and could speak from personal knowledge of the adequacy of the plant and of the care taken by the staff of the Dominion Bridge Company and the Consulting Engineers to secure consistent results.

Mr. Pratley. Mr. P. L. PRATLEY, of Montreal, was of opinion that, while the span was not so long or so heavy as to be of outstanding significance or to warrant unusual refinements in theoretical treatment, the physical and climatic conditions, the various limitations imposed by economic and political considerations, and the very definite place the project had been intended to occupy and did occupy in the development of Canadian bridge-building, added largely to the technical interest of the Paper.

No very satisfactory explanation had ever reached him as to why British wire-makers could not be interested in quoting for the 400 tons of wire required for the cables and suspender-ropes. They had apparently agreed that they would have experienced no difficulty in meeting the specifications or the delivery dates ; they knew that preference would be accorded to Empire products, and that it was unlikely that the products of Canadian wire-mills would be accepted, due to lack of experience, as being from "works of established reputation for the kind and character of wire specified." A Canadian wire-mill had voluntarily produced samples of cable-wire for experimentation and test, although the firm knew that they had been virtually ruled out of the present competition ; they had been anxious to be admitted for the next order, and so satisfactory did the engineers find their material to be, although it was not entirely up to the elastic requirements, that that mill had been encouraged to continue experimenting and to bid for the suspender-rope wire. They had done so successfully, having furnished first-class material, as the tests referred to by the Author on p. 388 would prove. The wire for cable-strands came from eastern plants of the United States Steel Corporation (the American Steel & Wire Company) located in Massachusetts and New Jersey, and constituted the only non-British material in the bridge, as the billets for the Ontario-drawn suspender-rope wire were of Scottish origin.

It might also be mentioned that the five hundred and ninety-five roadway stringers, which had been ordered as 13-inch N.B.S. Beams, and detailed as such, had all been found to be 13¼ inches deep on arrival in Canada, in spite of mill-inspection at Skinningrove ; changes were therefore necessary throughout all the drawings. To

allow for probable diversity in origin, the carbon steel for the stiffening trusses had been required to conform with either the B.E.S.A. Specification for Structural Steel for Bridges, or the Canadian National Railways Specification for Special Carbon Steel for Bridges, or the engineers' own specification for the Montreal Harbour bridge, 1925, all of which were substantially equal as to physical and chemical requirements. Actually, the Canadian National Railway Specification SW 1.1/1928 was incorporated in the mill orders to all the firms in Great Britain who supplied steel. Mr. Pratley.

Much study had been given to the truss-type before panel-lengths, depths between chords, web-bracing system, and build had finally been adopted. It had been desired to use available stringers economically, and as the market trend had suggested the British Standard Beams, which were much more limited in number than the American standards, the choice of stringer-profile had led to an optimum panel-length and depth of truss. Several combinations of panel-lengths and web-arrangements had been fully sketched with a view to securing the minimum weight of material for details, and the minimum over-all weight. The transverse stiffness of the upper chord, in view of the absence of top laterals or sway bracing, had also been given prolonged consideration. The 8-foot panels mentioned by the Author referred to the top chord only. The verticals were 16 feet apart, but intervening diagonals met the chord at intermediate points. The chords had been designed with their flange-angles turned outwards, and the vertical members had been provided with continuous solid web-plates and knee-gussets to the floor-beams as additional means of furnishing lateral support. Even then the working unit compressive stress in the upper chords had been arbitrarily reduced by 2,000 lbs. per square inch from the figure obtained for N.T.W. loading under the formula $\frac{28}{18}\left(17{,}000-60\frac{l}{r}\right)$. The majority of the upper chord members had thus been proportioned for a unit stress of 20,800 lbs. per square inch on the gross section, as the governing radius of gyration was 4·95 inches. As a further contribution to lateral stiffness, the upper chords were well and continuously latticed on both faces as compared with the batten-plate system adopted for the lower chords. Continuity of truss-framing throughout the suspended structure was only given scant consideration, as no economic advantage was procurable on so short and light a span, and expansion troubles would have been much aggravated by a continuous-span construction.

The dominating factor in the choice of the light "Teegrid" floor was the all-round reduction in weight, and the consequent relief in horizontal pull at the bases of the anchor-piers. Careful figures

Mr. Pratley. made previous to the call for tenders had served to establish a unit-price per pound, or per square foot of "Teegrid" floor, which would avoid excess cost over the more normal type of reinforced-concrete floor slab of about 6½ inches in thickness without any special wearing-surface. Inquiries into the probable cost of welded "Teegrid" construction, which in its original form had been more or less a patented device of the Truscon series of industrial products, but which had been modified slightly to meet Mr. Pratley's conception as to how its component parts might function, had led to the belief that the critical price would hardly be met by manufacturers, even were all the incidental erection advantages fully reflected in the figure tendered to the Provincial Government. It had been regarded as likely that the over-all excess cost consequent upon the adoption of the "Teegrid" floor might reach some $7,000 (£1,400 at current rate of exchange) after making all reasonable allowances for reductions in cross beams, cable-wire, and towers, but he was satisfied that the actual contract price reduced that estimated excess almost to nil. The test grids mentioned by the Author on p. 363 had been watched with considerable interest. An effort had been made to determine mathematically the system by which the loads applied through the imitation truck-wheels would distribute themselves longitudinally and laterally, and so to forecast the point at which failure might be first expected. The distribution as recorded during the test had seemed to be distinctly better than that suggested by the calculations, but the first metal failures had occurred as anticipated in the 2-inch welds at 20-inch centres between the toes of adjacent T-sections. Those welds had not broken, however, until the total load had reached 50 tons, when two of them situated directly under one of the wheels had cracked simultaneously. At that stage the test-slab had been distinctly bowed transversely, the maximum deflection having been about ¾ inch. Upon that initial failure the load had been released, and perfect recovery of shape appeared to have taken place. Upon a new application of a 50-ton load three other similar welds had given way, and the test had been discontinued. As far as could be seen by the naked eye, or through a small magnifying glass, the bond between concrete and steel had nowhere been broken.

He thought that the bases and saddles shown in *Figs. 17, 18* and *19* constituted excellent instances where structural welding was distinctly advantageous in bridge construction. To those interested in that modern development it might be mentioned that heavily-coated electrodes had been used for the ½-inch and ⅝-inch fillet welds, and jigs had been provided so that each piece could be positioned as ideally as practicable for the actual welding process.

Mr. Pratley.

The bases had first been assembled in three pieces each, each piece consisting of the vertical webs only, one central section, and two wing sections. Those three sections had then been welded together, after which the upper plate, previously burned out to shape, had been connected; finally the heavier solid lower plate had been welded to the previous assembly.

At several points in the Paper the Author had referred to the fact that accuracy in shop-work had been definitely and profitably depended upon during the erection of the structure. For a bridge of that type, and particularly for such a site, it was essential that the maximum degree of accuracy be obtained in all shop-work, that the assembly of adjacent parts and the match-marking of them be liberally and faithfully carried out, and that the inspection be of a high order. Contractors and engineers alike benefited from such a practice, for the skimping of shop-work would always result in unnecessary field troubles, extended delays, and additional expense to all parties. Superior craftsmanship and intelligent supervision should be definitely encouraged and properly recognized.

There was no reason to doubt that careful measuring and marking during the pre-stressing operations could be accepted as providing ample accuracy in the setting of cable-strands, once the correction for the actual span between the towers had been made. A great saving in time and expense could be accomplished in the field if observation by instruments for adjustment could be eliminated, as experience at the Island of Orleans bridge would suggest to be the case. The precise form in which the mathematical properties of the catenary were stated on pp. 401 and 402 was a convenient arrangement if properly interpreted, and was due, as far as Mr. Pratley knew, to the engineering staff of Messrs. Robinson and Steinman, Consulting Engineers, of New York.

After the Paper had been written he had carried out certain observations and tests on the finished structure, some brief mention of which might be of interest. Early in the morning of 26 September, 1935, the "lean" of the towers had been measured under ideal weather conditions, with no sun, no wind, and a normal temperature of exactly 60° F. About 9.30 a.m. a group of heavily loaded gravel trucks had been assembled at the centre of the middle span, and the resulting vertical deflection of the central floor beam had then been measured. The towers had been found to lean southward $1\frac{11}{16}$ inches and $\frac{7}{8}$ inch on the south and north sides respectively, the discrepancy being most probably due to the prevailing winds which had persisted during erection; the deflection, calculated as $5\frac{7}{8}$ inches for the 43-ton load, had been recorded as $6\frac{3}{4}$ inches. It was felt that that excess was due entirely to the inaccuracy of the fundamental

Mr. Pratley. assumption, adopted in the theoretical calculation, that all live loading on the span was uniformly distributed by the truss and suspender system to the cables. It was quite evident that the concentration of 43 tons on the middle 48 feet was a condition where that assumption could hardly be expected to be fully realized, and the check between the measured figure and the calculated deflection was therefore regarded as very satisfactory.

Mr. Toms. Mr. A. H. TOMS observed that the use of welding for the fabrication of tower saddles and shoes had eliminated many of the doubts regarding soundness which were felt even with the best castings; provided great care was taken in the organizing of the welding and normalizing procedure, initial temperature stresses ought to be of but small consequence. He was, however, of the opinion that, where possible, the fabrication of such structures from forged slabs and rolled plates and sections riveted together was possibly less open to objection from those points of view than either castings or welded structures.

There could be no doubt as to the desirability of pre-stressing the cables and suspenders, for, by thus raising the modulus of elasticity and bringing about a uniformity of the mechanical properties of the cables, a definite economy in material and a greater precision of work was possible. It was there that ropes of the "locked-coil" type showed a definite advantage. The principle of pre-stressing structures was far from new, and it had been used in various ways with varying degrees of success for such purposes as the reduction of secondary stresses in bridges, the strengthening of dams, and more recently in combination with the process of compacting concrete by pressure and heat in the production of remarkable concrete structures. The value of the process of pre-stressing did not, however, appear to be as generally appreciated as it should be.

Possibly the most interesting feature of the bridge was the "Tee-grid" floor-system, which had the advantage of lightness, that being of paramount importance in a long-span bridge, since every pound per square foot reduction of floor dead load was reflected in a cumulative reduction in the dead load of the main structure. It was stated in the Paper that that type of floor had not been adopted until it had satisfactorily passed tests to destruction, but it would be of interest to know how long those test sections of floor had been cast when tested, since it would appear at first sight that any shrinkage of the concrete filling would destroy the bond between the concrete and the steel tees and would thus upset the action as a composite section. Another point which seemed to be of importance was the possibility of the vibration from the passage of vehicles assisting in the destruction of that bond and also in the displacement

Mr. Toms.

of the half-round cross-bars. In view of the all-welded nature of the floor system, and the consequent difficulty of replacement of the decking, absolute certainty of the soundness of the type of construction appeared to be essential. He would be glad to receive any further information which might be available on those points.

Mr. Woodruff.

Mr. G. B. Woodruff, of San Francisco, observed that there had been no recent development in bridge engineering more interesting than that of the application of the suspension type to crossings when a comparatively light long-span structure was required. That type had permitted the construction of a bridge at points where more expensive cantilever or arch designs could not be economically justified. In each of the successive designs set out in Table I, several improvements on past practice had been made. In the Island of Orleans bridge, among the innovations were welded members for bases and saddles of the towers and the new design for roadway expansion joints.

In any structure where the designer was not bound by rigid specifications the question of live loads and unit stresses would always arise. In the Island of Orleans bridge the congested load of 1,200 lbs. per linear foot might be assumed to be made up of 500 lbs. per linear foot, or a continuous train of heavy trucks, for each roadway-lane, and 20 lbs. per square foot of sidewalk. In any location, considering that trucks formed less than 20 per cent. of vehicular traffic, such a load was improbable, but to cover any contingencies, the assumption of such a load for the main carrying elements (cables, towers and anchorages) was most prudent. On the other hand, in designing the stiffening-trusses, the probability of such a load on half the main span with no live load on other portions of the structure, or other similar critical loading conditions, was so remote that an assumed loading of half that specified ought to prove safe. The unit stresses used were in agreement with usual practice, except that the value of 19,000 lbs. for the combination of direct load and bending for the silicon-steel towers might be considered ultra-conservative when compared with practice in some other structures.

As usual on other long-span bridges, a light floor had been adopted and, for that purpose, "Teegrid" had been chosen. No more severe exposure could have been found for such an installation, and the service records of that floor would be of interest to all bridge engineers. The explanation given by the Author to account for the top-chord participation in the lateral stresses appeared to be complicated. As the truss deflected laterally, the bottom chords would elongate on one side and would shorten on the other. The truss

Mr. Woodruff. diagonals would force the top chords to participate in those elongations. That action would produce stress in the diagonals, which action, however, would not be that of shear in the stiffening truss.

It might be of interest to compare the make-up of the sections used with those that would probably have been adopted had the bridge been designed for fabrication in the United States. In the latter case the floor beams and truss verticals would have been rolled sections, and rolled channels would have been used for the chords. The towers would probably have been composed of rolled sections riveted together, although that would have involved some sacrifice in the silhouette of the towers. Those who had inspected steel castings would applaud the welded design for the tower-bases and saddles. The care exercised in normalizing the stresses after welding was noteworthy, and was typical of the caution used throughout the work.

He had confined his remarks to questions of design. The Author had completely described the interesting erection processes, and the Appendix contained the best description of rope and strand prestressing that had come to Mr. Woodruff's attention. The Author deserved the thanks of the profession for the instructive manner in which he had presented the entire subject.

The Author. The AUTHOR, in reply to the Discussion and Correspondence, observed that, on account of the wide scope of his subject, he had found that it had been practically impossible to cover every aspect of the bridge construction in the Paper, and he was therefore particularly indebted to Mr. Pratley and Mr. Armstrong for the additional information which they had provided, including details regarding the test that had been carried out after the completion of the structure.

There had been several comments made on the subject of the loadings adopted for the design. While those loadings were small in comparison with the requirements of some standard specifications, he was of the opinion that, in view of the conservative maximum stresses allowed, and bearing in mind that very peculiar arrangements of live load were required to produce maximum stresses, the structure was perfectly capable of carrying the ordinary road traffic of that part of the country. In that connection, Mr. Woodruff had pictured the " congested " loading as equivalent to that of a continuous train of heavy (about 4-ton) trucks on each roadway-lane, together with a side-walk load of 20 lbs. per square foot. Mr. Portas had referred to the disparity between loading-requirements in England and in Canada. That, the Author thought, was due to the high efficiency of the highway-system of the former country, and the consequent possibility of the movement of extremely heavy loads by road.

The Author.

In answer to Mr. Roberts, he observed that no specific tests had been made regarding the live-load concentration to be anticipated, but there was no doubt that the concentration assumed was entirely adequate. In reply to Mr. Gill, the assumed wind load of 300 lbs. per linear foot of the bridge was equivalent to a load of 30 lbs. per square foot applied to one-and-a-half times the area of the vertical projection of the truss. In addition to that allowance, a load of 100 lbs. per linear foot had been assumed to act upon the exposed surfaces of the vehicles in the case of the "normal" loading. In view of Mr. Armstrong's opinion that the cable-design had been too conservative, and of Mr. McConnel's reference to the recent use of higher cable-stresses, he thought it would be of interest to mention that he was at present working on the design of a 1,500-foot suspension span, and that a maximum tensile working stress of 90,000 lbs. per square inch had been proposed for the main cables.

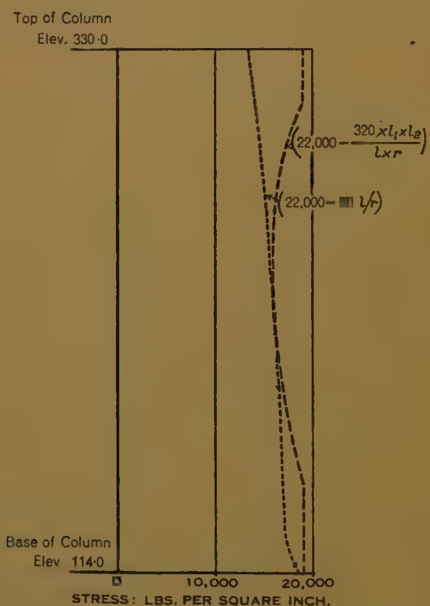
Attention had been also drawn several times to the formula (p. 379) which had been used in the design of the tower-columns. The function of the many column-formulas which were in common use was to limit the allowable fibre-stresses on account of the tendency of a compression member to buckle. Those formulas, as they did not usually differentiate between the conditions at the ends and at the centre of the member, were adapted chiefly to the design of comparatively short struts and columns of constant section, such as were found in trusses and building-frames. The effect of the empirical formula in question was to permit a higher unit-stress at the restrained ends of the tower, while still imposing the usual limitations at sections near the middle of the length, where the secondary stresses due to buckling were of most consequence. In *Fig. 43* (p. 456) Mr. Pratley's formula was compared diagrammatically with the conventional formula $f_c = 22{,}000 - 80\ l/r$ lbs. per square inch, to which it was related. For the purpose of the comparison, both formulas were applied to the column sections (p. 381) adopted in the design, and it would be noted that both gave the same permissible stress at the mid-point of the length. Apart from the question of limiting stresses, the column had, as Mr. Armstrong had stated, been designed to deal with all eventualities of loading. He thought that Mr. Gough had, in his comments, confused bending- and buckling-allowances.

Mr. Woodruff had made an interesting comparison between the sections used for the floorbeams and truss-members and those that would probably have been used if the fabrication had taken place in the United States. Unfortunately for the Canadian engineer, protective tariffs during the past few years had very largely prevented him from taking advantage of the economies which frequently

The Author. derived from the use of heavier rolled sections in preference to members built-up from smaller material. With regard to the material used for the main towers, the use of "Chromador" steel had been permitted since that steel filled the requirements of the specification, although the latter had in the first case called for the use of silicon steel.

The precise economical sag/span ratio of a suspension bridge was a quantity that was not easily determined. It depended upon a multitude of factors, among which could be cited the foundation conditions obtaining at the site, the capacity of the bridge, and the

Fig. 43.



fluctuating unit prices for the many processes and materials of construction involved, accurate estimates of many of which only became possible subsequently to the call for tenders. An increase in the ratio, while generally producing an economy of material in the main supporting members of the structure, had the effect of reducing both the vertical and the lateral rigidity, and at the same time the rapidly-mounting cost of the tower-erection became more significant. The selection of the ratio depended, too, on æsthetic considerations. The ideal ratio had been variously estimated in the past as lying between the limits of 1/6 and 1/12 ; the experience of modern practice

had tended further to define this range, and limits of about 1/8 and 1/10 could be considered as generally accepted. The cost of the suspension bridge had been found not to be materially affected within the range.* The Author.

Mr. McConnel and Mr. Portas had drawn attention to the modern tendency towards shallower stiffening-trusses. As Mr. Portas had said, the function of the truss was to limit the local deflections of the suspended platform, and not to relieve the cable. The trusses were not designed to support any dead load (except locally, between suspender-points), and, in the case of the present bridge, they carried only 1 or 2 per cent. of the live load. As the span-length increased, the size of the truss became of less significance. On the recently-constructed George Washington bridge † there was no truss at all (though provision had been made for the construction of trusses, together with a second deck, when the traffic requirements became sufficiently large). The question had been recently discussed in an interesting manner by Mr. H. E. Wessman.‡ The stiffening-truss as first assumed for preliminary design of the Island of Orleans bridge had had a depth of 13 feet 6 inches, with moments of inertia in the central and side spans respectively of 4,600 and 3,500 inch^2-feet^2 units. The assumed cable-area had been 49·5 square inches. The Author had found that the bending-moment in the truss as designed by the "elastic" method exceeded that computed by the "deflection" method (and using the same data) by about 54 per cent. Regarding the economies consequent on the adoption of a lighter deck, he had intended to convey the idea of a general reduction in the weight of the whole superstructure. In the isolated case of the stiffening-trusses, he was aware that, while there was a saving of material in the floor-supporting members, there was also an increase in the size of the chords, owing to the greater flexibility of the suspension system.

He agreed with Mr. Gough that it was incorrect to increase the moment of inertia of the trusses on account of the effect of the web-system alone. The effect of the chord-details (which included splice-material, vertical and horizontal gussets, lacing-bars and battens, amounting in all to more than 30 per cent. of the weight of the chord-material proper) and floor-system, and of the stiffness of the riveted joints, however, was in the opposite direction. The conventional figure of 8 per cent. was designed to take both those effects into consideration.

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* G. A. Hool and W. S. Kinne, "Movable and Long-Span Steel Bridges," p. 326. New York, 1923.

† Trans. Am. Soc. C.E., vol. 97 (1933).

‡ *Discussion on* "The Relation of Analysis to Structural Design." Proc. Am. Soc. C.E., vol. 62 (1936), p. 430.

The Author. The maximum calculated deflection of the central span was 4 feet under the "congested" loading, and 3 feet under the "normal" loading, at the normal temperature of 60° F. Temperature deflections (which could be added to or subtracted from the above figures without involving any significant error) were 1·6 foot downwards (at 120 degrees F.) and 2·2 feet upwards (at — 20 degrees F.). The maximum displacements of the main saddles would be found on p. 379. Attention might also be drawn to the observed deflections referred to by Mr. Pratley on p. 451. With regard to the partial loading which was considered by Mr. Roberts, the Author's figures agreed fairly closely with those of *Figs. 40* and *41* (pp. 431 and 432). In the case of *Fig. 40* (which depicted the case of loading giving the worst grade of the roadway), however, the point of zero deflection was approximately at the quarter-point of the span, and not as was shown in the diagram. The effect of the unequal deflections of the two trusses and cables was to put a small amount of bending into the vertical members, owing to the eccentricity of the suspender pull, but at the same time there was a relief of direct stress since the loading was only partial. In his opinion, the stiffening-trusses of a suspension bridge should be strong enough to limit the maximum grade in the bridge, under the worst combination of loadings, to about 4 or 5 per cent. Modern automotive traffic could cope easily with such grades, but it should be borne in mind that in many localities there was still, and would continue to be, a considerable amount of horse-drawn traffic. On the Island of Orleans bridge the winter sleigh-traffic had also to be catered for. The case considered by Mr. Roberts in *Fig. 40*, in combination with, say, zero temperature, would represent a most unlikely condition, but even in this case the grade at the end of the central span would not exceed 4·8 per cent.

In reference to the distribution of the wind load between the trusses and the cables, Mr. Portas was right in assuming that the chords participated in proportion to their areas. The Author was indebted to Mr. Woodruff for a more succinct exposition of the theory of upper-chord participation in the lateral loads, but he thought, nevertheless, that any diagonal-stresses, whereby were produced chord-stresses of intensity varying along the span, would correctly be described as shear stresses. In answer to Mr. Wilson, as far as his knowledge went, no measurements had ever been taken of the lateral deflections of long bridges due to wind force, although he was aware that in the past it had been found necessary to provide lateral guys to restrain those movements on certain suspension bridges. Reverting to the distribution of wind loads, which, contrary to Mr. Wilson's impression, were not resisted entirely by the stiffness of the platform, he would refer him to a very

The Author.

complete treatment of the subject which had been published in 1932.[1]

The subjects of æsthetics and of general economy of design had been considered by Mr. Legget and Mr. Portas. With regard to the former subject, he felt that the suspension type of structure, with its inherent grace of line and proportion, could scarcely be considered as out-of-place in any location. It should not be thought, however, that in that statement he implied an adverse criticism of the appearance of other types of bridges, of which there were to be found innumerable instances that were in splendid harmony with their surroundings. In point of economy there was no doubt that, as Mr. Woodruff and Mr. Portas had observed, the suspension bridge, on account of its use of so much material in pure tension, was regarded as very well adapted for bridges of medium and long spans. As he had stated in the Paper, several other types of bridge had been considered by the engineers before the decision was made in favour of the design adopted. Careful estimates had been prepared for designs of those alternative types. In each instance, the estimated cost for the case of a main span opening of 600 feet clear between piers had come to within 3 per cent. of the cost of the structure as it had been built. In the case of a main span providing the required 600-foot width of full vertical clearance, however, the estimates had indicated costs of about 3 per cent. in excess of that of the adopted design. Apart from the economy of the design, there was hardly need to point to the advantage of the 1,000-foot clear span, as built, over a 600-foot span.

Adverting to the more particular case of the æsthetic treatment of the main towers (Figs. 15, Plate 1), the design had been the subject of a great deal of preliminary sketching and of much thought by the consulting engineers. The adopted layout departed from a purely utilitarian design, not only in regard to the arching of the portal and of the top strut, but also in that the bracing members had, at some extra expense, been considerably increased in width, the object in each case having been to render the structure more pleasing to the eye. In regard to the "functional" aspect of the design, the arching referred to might be considered as producing an effect complementary to that of the inward leaning of the tower-legs. A further refinement, which had not been mentioned in the Paper, but reference to which might be of value, had been the application of entasis in the other elevation of the towers (Fig. 15, Plate 1), by the employment of two distinct batters on the outer

[1] L. S. Moisseiff and F. Lienhard, "Suspension Bridges under the Action of Lateral Forces." Trans. Am. Soc. C.E., vol. 98 (1933), p. 1080.

The Author. surfaces of the side box-sections.[1] The slope of those surfaces changed, at elevation 198·75, from $\frac{1}{90}$ in the part of the tower below that elevation to $\frac{1}{63}$ in the upper part of the tower.

Mr. McConnel had drawn attention to the Florianopolis bridge, Brazil, the unique design of which had been very fully described and discussed [2] some years ago. He could not, however, agree with the comparison which Mr. McConnel had drawn between that bridge and the Island of Orleans bridge. He thought, in the first case, that the "congested" loading of 1,200 lbs. per linear foot should have been quoted rather than the "normal" loading of 900 lbs. per linear foot, since the heavier load had been used in designing the cables and towers. It might also be mentioned that the eye-bar cables of the Brazilian bridge had been designed to a live loading of 1,850 lbs. per foot.

Regarding the comparison of weight, he thought that in view of the fact that the cable did not carry the side-spans in the case of the Florianopolis bridge, a fairer comparison would result if the weight of the trusses, floor, and suspenders in the side-spans were subtracted from the Island of Orleans total. He would draw attention, too, to the fact that the total length of the suspension system was 2,370 feet in the case of the Island of Orleans bridge and 1,919 feet (but with a central span of 1,114 feet) in the Florianopolis bridge. He had compared the weights, on the foregoing basis, as follows :—

	Island of Orleans.	Florianopolis.
Cables, complete	436 tons	780 tons
Suspenders	12 ,,	5 ,, (about)
Towers	630 ,,	920 ,,
Anchorages	144 ,,	110 ,,
Trusses and bracing (central span only)	570 ,,	840 ,,
Floor framing (central span only) .	290 ,,	420 ,,
	2,052 tons	3,075 tons

He felt, however, that further calculations would have to be made before a true estimate of the relative economy of the two types of structure could be achieved. Mr. McConnel, in his reference to many different bridges, had dealt with instances which varied widely in regard to size, capacity and other special circumstances governing design. The Author thought that, except in the case of shorter

[1] p. 382.

[2] D. B. Steinman and W. G. Grove, "The Eye-Bar Cable Suspension Bridge at Florianopolis, Brazil." Trans. Am. Soc. C.E., vol. 92 (1928), p. 266.

bridges, any economy of material which might be achieved by the use of rocker-towers was outweighed by what was, in his opinion, the very unsatisfactory appearance of the articulation at the bottoms of those towers. In connection with the separation of the strands of a cable, he observed that that form of construction was only feasible when there was but a very small number of strands to be dealt with, and that, if the strands were wrapped, that operation would be relatively much more expensive. The Author

Professor Lea's remarks about the fatigue properties of cold-drawn bridge-wire, and the diagrammatic representation (*Fig. 38*, p. 424) of the results of some of his research work, were of great interest. Although the Author had not specifically mentioned the amount of the dead load in the cables, it was implicit in the data given regarding dead loads and cable-sags. It was about 56,000 lbs. per square inch, that was to say 75 per cent. of the maximum stress, which was 75,000 lbs. per square inch (D.C.T. combination of loading). The corresponding fatigue range, obtained from *Fig. 38*, was 66,000 lbs. per square inch, and the factor of safety was $\frac{66{,}000}{19{,}000} = 3{\cdot}48$. The maximum stress due to the D.N.T. combination (which would probably never be reached) was about 71,000 lbs. per square inch, and the factor of safety in that case came to 4·4. For the condition illustrated in *Fig. 39*, the factor of safety became 1·16. He had been unable to check Professor Lea's computation on p. 425 showing that the factor was 2 for a dead-load stress of 45,000 lbs. per square inch. By comparing similar triangles in the diagram he thought that the dead-load stress for a factor of safety of 2 should be about 39,000 lbs. per square inch. Incidentally, while the subject of the factor of safety of the cables was under consideration, he thought it well to mention that the average tensile strength of the wire had been in the neighbourhood of from 225,000 to 230,000 lbs. per square inch, whereas the specified strength of 220,000 lbs. per square inch had been used as the basis of the foregoing discussion. The various factors of safety were thus actually somewhat higher than those computed above.

Adverting to the question of " creep " on the saddles, he had never come across any reference to that possibility. It did not seem at all likely to him, as the high towers were extremely flexible, and consequently exerted very small longitudinal forces on the cables. Also there were eight keeper-castings on the saddles which would assist in preventing movement. He could, however, imagine a growth or contraction of the cable-strands relative to the grooves on the saddle, due to changes in stress, but those movements at the worst would be of the order of 0·02 inch and would take place very

The Author. gradually. Also, the full bending effect would apply to the bottom strands only. There would be no movements due to temperature, as Mr. Webster had said on p. 433. In any case, the Author thought that, since the accumulation of stress-repetitions would take place over a long period of time, the consequent fatigue of the metal would not be comparable to that which might be caused by rapid fluctuation of stress. He concurred with Professor Lea's opinion that a continuously-changing curve in the grooves of the saddle would be a great improvement, but he believed that the accurate fabrication of a saddle with such a curve would be an impracticable operation.

Professor Lea had been mistaken in assuming that the yield-point of the wire was defined as occurring at an elongation of 0·075 per cent. That elongation was stated correctly in the Paper as 0·75 per cent. Referring to *Fig. 37* (p. 423), and taking an ordinate of 175,000 lbs. per square inch (that being the average yield-point stress of the wire used for the Island of Orleans bridge), it would be seen that the corresponding strain in the case of Professor Lea's specimen was about 69×10^{-4}, or 0·69 per cent., which was a quantity comparable with the 0·75 per cent. of the definition.

In view of the trouble which was experienced in 1929 during the erection of the Mount Hope bridge, in consequence of which the cables of both that bridge and the partially-erected Detroit bridge were removed and replaced by cables of cold-drawn wire, the use of heat-treated wire had not been considered for the strands of the Island of Orleans bridge. He wished to correct Mr. Gill's impression that the wires at Mount Hope had broken on the main saddles. It was at the points of tangency with the strand shoes at the anchorages that the fractures had occurred. On the Detroit bridge (with the design of which Mr. Pratley had been intimately connected) such broken wires had not been found, but the cables had nevertheless been dismantled. A paper[1] by members of the staff of the U.S. Bureau of Standards had recently been published, giving the results of a 6-year investigation into the causes of the Mount Hope failures. Mr. Gill had commented on the remarkably high efficiency of the cable-strands. It would be noted that Mr. Meals had obtained a comparable figure by calculation (pp. 441 and 442). The Author agreed with Mr. Armstrong that the high efficiency was largely due to the very positive bond between the inner wires, which resulted in a uniform distribution of stress among the wires. That bond might to some extent have been assisted by the effect of the two-directional lay.

The interesting subject of the use of "locked-wire" strands had

[1] Read by W. H. Swanger and G. F. Wohlgemuth at the annual meeting of the American Society for Testing Materials, 1936.

been mentioned, and some possible advantages deriving therefrom had been listed. As the Author understood the matter, however, a "locked-wire" cable would still consist of thirty-seven strands of approximately $1\frac{3}{8}$ inch diameter (as shown in Fig. 7, Plate 1) and the use of some kind of filler to round out the section would still be necessary. In addition to the two disadvantages mentioned (the higher cost of the strands, and the reduced efficiency of the cable-band assembly on account of the smooth surface of the strands), the Author was of the opinion that there must be some weakness inherent in the trapezoidal and angular cross sections of the outer wires. He imagined that the ordinary circular cross section would possess greater strength on account of its symmetry and the consequent simplicity of stress-distribution in the case of a possible variation from a purely tensile loading. It had been argued that the "locked-wire" strand was less liable to corrosion, but, on the other hand, experience had shown that an adequately-wrapped cable, even of ungalvanized wires, was very little subject to that influence. Regarding the cedar-wood fillers, they were (before being impregnated with oil) shaped to fit the profile of the outer strands of the cable, and they almost completely filled out the voids between the strands (Fig. 7, Plate 1). The Author.

The Author thanked Mr. Meals for drawing his attention to the incorrect representation of the strand make-up in Fig. 8, Plate 1, and for the correction shown in *Fig. 42* (p. 443).

Dealing with the socketing of the cable-strands, the point had been raised by Professor Lea that the properties of the wires might have changed after being heated to 850° F. during the pouring of the zinc. It could be assumed that they were unaffected by that process, as the temperature of the galvanizing bath had been from 875° F. to 900° F. The Author, to convince himself, however, had made some tests on pieces of wire that had actually been removed from a socket, and could find no apparent change in their tensile strength. Mr. Gill had questioned the advisability of vibrating the sockets during pouring. That was done to assist the flow of the metal and to prevent the formation of air-pockets, and was, in the Author's opinion, essential to a satisfactory pour. The hand-hammering was stopped as soon as the zinc showed signs of setting, and the socket was then left in position in the apparatus, and not moved for at least 30 minutes. Subsequent examination of the moulds from several of the temporary sockets,[1] some of which were sawn across for the purpose, showed an eminently satisfactory penetration of the zinc, and there were no signs whatever of cracking.

[1] P. 418 of the Appendix.

The Author. Contrary to Mr. Hamilton's impressions, "slip" was noticed when the sockets were under load. During the first application of load (while pre-stressing) a slip of about $\frac{1}{8}$ inch occurred in every socket. That took place during the application of the first 10 tons of load, the wires and the zinc moving integrally inside the steel forging. No subsequent movement was ever noticed. The effect of repeated loading was not studied in itself, but, during the repeated loading incidental to pre-stressing and to tensile-testing in the laboratory, none of the sockets evinced any sign of distress.

He was in agreement with Professor Lea that the weakest point of the suspender-rope was undoubtedly at the point of tangency of the rope on the cable-band. Mr. Meals's remarks had indicated that the extent of that weakness was largely conjectural and that there was a great need for further research on the strength of stationary wire ropes in combined tension and bending. The suspender-ropes, however, had been designed with that in mind, and very conservative designing-loads and stresses had been used. The type of rope selected (*Fig. 22*), with wires of small diameter, was that most suited for bending over a small radius. It was unlikely that any movement would ever occur at the cable-bands, since equal loading of the rope on either side of the band was ensured by the careful establishment of a centre-mark, and also because of the magnitude of the loading. Referring to Mr. Gill's comment on the absence of adjustment for the suspenders, Mr. Wilson had stated the case very clearly. It was impossible to make an accurate measurement of a rope when it was unloaded or subjected to an unknown tension. The omitting of adjustment, either by turn-buckles or by shims, was a comparatively recent development in design, and, as far as he knew, the positive pin-connection of the suspender of the truss had been previously used only on the Waldo-Hancock bridge, Maine, U.S.A., which had been designed by Messrs. Robinson and Steinman. Regarding the accuracy of the suspender-rope (and strand) lengths, the calculations in the case of the Island of Orleans bridge were performed and checked independently by both the contractors and the engineers, and perfect agreement of the two sets of results was arrived at before any cutting of the ropes was commenced.

Reverting to Mr. Meals's remarks, the advantage of using a Lang-lay independent wire-rope centre would lie in the fact that the contact between adjacent wires would be in the nature of "line" rather than "point" bearing. While that arrangement was undoubtedly the means of increasing the length of the life of a moving rope, it had much less significance in the case of a fixed suspender-rope, the capacity of which was not affected by the direction of the

The Author.

lay of the core. Dealing with Mr. Gill's inquiry as to whether the brazed joints in the suspender-rope wires were satisfactory, the Author had no hesitation in answering in the affirmative. While it was true that the efficiency of a brazed splice was comparatively low,[1] it should be remembered that there were one hundred and sixty-three wires in the cross section of the rope, and that consequently the reduction in strength of the rope due to a brazed splice was of the order of $\frac{1}{3}$ per cent. It might be mentioned that care was taken to keep such splices as far apart as possible, so that there would be no further reduction in strength.

Professor Lea was correct in stating that the steel of the "Teegrid" slab was subject only to low stresses, of the order of 7,000 lbs. per square inch. The Author, however, could not quite understand his reference to the welded connections of the "Teegrid" to the stringers as points "definitely . . . of real weakness. . . ." Those particular welds were not "strength" welds, their function being merely to hold the grid-sections (which were self-supporting and would be so even without the welds) in position and to prevent any movement under traffic. A careful inspection of the slab, carried out in May, 1936, showed that the paint on the underside was in excellent condition, with very few indications of percolation of water from above. The wearing surface was reported as being "in just as good condition as when built," although in that connection he thought it right to mention that the traffic had not been heavy. With regard to the bond between the steel and the concrete, referred to by Mr. Toms, precaution was taken during the pouring of the concrete (by the use of a dry mix, vibrated) to ensure good contact between the materials. There was no indication whatever of the concrete failing in the neighbourhood of the stems of the tees or of the half-round cross-bars. In fact, it was anticipated that the surface steel would have the effect of preventing the wear of the concrete, and it would be noticed that Mr. Moisseiff had, in the course of his remarks on the subject of composite steel-and-concrete slabs, also subscribed to that opinion. The further performance of the slab would, in view of the foregoing discussion, be watched with great interest, the more so since, as Mr. Woodruff had observed, the climatic conditions at the site were very severe.

In connection with the use of asphalt as a wearing surface, experience on two large bridges in Eastern Canada (the Ambassador bridge, at Windsor, Ontario, and the Jacques Cartier bridge, at

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[1] The Author had conducted tensile-tests on several random specimens of brazed wires of different diameters, and found the average efficiency to be 54 per cent.

The Author. Montreal) had shown that that type of surface had cracked very badly, probably under the influence of wide variations in temperature. Water had collected in the cracks, and, freezing, had further destroyed the asphalt, and in some instances had even caused the partial disintegration of the concrete slab. The use of asphalt was now definitely discouraged by the consulting engineers. Furthermore, the weight of an asphalt wearing-surface would have rendered uneconomical the use of the "Teegrid" slab.

He would refer Mr. Legget to pp. 449 and 450, as the information given there by Mr. Pratley seemed to answer very fully his queries regarding the origin and tests of the slab. Adverting to Mr. Toms's request for further information regarding the test-slab, he stated that the concrete used had been made with the Canada Cement Company's "XXX" quick-setting cement. Eight 6-inch test-cylinders were made at the time of pouring of the test section. Tests were made on the 9th, 10th and 11th days after pouring, and the test-cylinders, which were broken on the same days, showed strengths varying between the limits of 3,800 and 5,100 lbs. per square inch. The concrete used in the field was of ordinary cement, and the routine tests showed strengths averaging about 3,600 lbs. per square inch at 8 days, and about 4,800 lbs. per square inch at 28 days.

With regard to the use of aluminium in suspension bridges, he did not believe that that material was yet available at a price which could justify its use on new constructions. He had, under the direction of Mr. Pratley, made some calculations for a bridge now being designed, and the results had led him to that conclusion. In the case of the proposed re-decking of the Brooklyn bridge, which Mr. McConnel had mentioned, the use of aluminium might be justified because, by the consequent reduction of weight in the floor and trusses, it would obviate the necessity for replacement of the remainder of the structure. Engineers, in general, were not yet familiar with the structural characteristics of that metal, and he would refer Mr. McConnel to two articles relating to the installation of an aluminium bridge floor[1] at Pittsburgh, Pa. Objections to the use of a timber deck were the fire-hazard and the cost of maintenance.

With reference to the hinged expansion-joints in the roadway, the chief aim of the designers had been to avoid maintenance trouble and expense, and as far as could be seen from the experience of last winter (1935–6), when there was no trouble, that aim had been

[1] J. P. Growdon, R. M. Riegel, and R. L. Templin, "Heavy Bridge Floor Replaced with Aluminium." *Civil Engineering*, vol. 4 (1934), p. 113. H. D. Johnson, Jr., "Aluminium Stringer Failures due to Fatigue Loading." *Engineering News-Record*, vol. 116 (1936), p. 318.

The Author.

achieved. Electric heaters would need regular maintenance and their initial cost (which would include the expense of housing and wiring and switchgear) would certainly exceed that of the mechanical parts of the joints now in use. In reply to Mr. Portas, the height of the curb ($6\frac{1}{2}$ inches) on the central portion of the bridge was determined by the need of conforming to that already in existence on the approach viaducts.

He wished to apologize for his inadvertent and evidently misleading use of the expression "normalizing" in connection with the heat-treatment of the pedestals and saddles subsequent to welding. Normalizing, as he understood it, was the process of internal adjustment, accompanied by complete stress-relief, which took place from temperatures of the order of 1,700° F. In members of the type and size of those under discussion, such temperatures were out of the question on account of the accompanying distortion. Stress-relieving, such as was performed on the members in question, and which was designed, as Mr. Hamilton had stated on p. 434, to reduce the residual stresses due to welding, was usually accomplished by ". . . heating uniformly to at least 1,100 degrees F. and up to 1,200 degrees F., or higher, if this can be done without distortion. The structure . . . should be brought slowly up to the specified temperature and held at that temperature for a period of at least one hour per inch of thickness, and should be allowed to cool slowly in a still atmosphere."[1]

A very complete description[2] of the welded fabrication of the tower-pedestals had been published subsequently to the presentation of his manuscript, and he was indebted to Mr. Legget for having drawn attention to that publication. In further reference to Mr. Legget's remarks, he had had no desire whatever to minimize the part played by the contractors in the successful accomplishment of the work. There had been close and friendly co-operation between the contractors and the engineers at all times, and several noteworthy details of the construction (including the spherical bearing-surface of the anchorage, the movable expansion-joint, and the welded members at present under discussion) had emanated therefrom.

Turning to Mr. Toms's remarks, he agreed that, as a general rule, the use of castings should be avoided as much as possible where definite strength-requirements had to be met. In the case of such heavy and unusually-shaped members as the pedestals and saddles, he was of the opinion that, in spite of the advantages which might accrue therefrom, it was not practicable to use riveted connections

[1] "Procedure Handbook of Arc Welding," the Lincoln Electric Company, p. 44. Cleveland, 1934.

[2] Footnote 1, p. 439.

The Author. in their construction. He felt that, in the welding of those members, a comparatively new engineering process had been applied in a construction to which it was singularly well adapted. In view of the care which had been exercised in the design and fabrication of those members, and of the nature of the loading to which they would be subjected in the structure, there was no reason to doubt that the confidence of the engineers and the contractors had been well placed in that application of welding.

Mr. Kennedy and Mr. Legget were correct in assuming that the first strand to be erected on each side of the bridge was the guide-strand. That strand did not differ from the thirty-six other strands with which it formed the cable, except in that it had been marked, during pre-stressing, with extraordinary care. No trouble of the nature suggested by Mr. Kennedy had been experienced. It was hardly to have been anticipated, as the tension in the freely-hanging strand was only about ½ per cent. of that to which the strand had been subjected at Longueuil. Regarding the effect of reeling and unreeling the strands after pre-stressing, he would refer Mr. Wilson to p. 420 where there was a description of the precautions taken to guard against trouble which might have arisen from that cause. Mr. Armstrong had made reference to the necessity for two re-adjustments of the cable-strands during erection. It should be pointed out, however, that both the disarrangements referred to had occurred only during the assembly of the first three layers of strands, and before the final and most satisfactory system of restraining jigs had been evolved. In each case the readjustment was readily performed, and the loss of time was not serious, as new strands were being pulled across the footwalks in the meanwhile.

In the matter of ice-sheathing, the Author, who was present at and had kept careful note of the cable erection, recollected that the coating of ice on the footwalks and strands persisted only for a few days, and that the open nature of the walkway deck offered a reasonably secure foothold during this period, provided that ordinary care was taken. He would recommend the same type of walkway, though perhaps of even lighter construction, for similar work in the future. As Mr. Armstrong had stated, the weather-conditions during the erection of the superstructure had been extremely severe, and the fact that adjustment of the strands had been made in so eminently satisfactory a manner was in itself a tribute to the skill and willing perseverance of the erection force which had worked under such trying conditions. In that connection, and having himself had experience, he felt that he could not stress too strongly the advantages, in the shape of greater comfort and consequently more accurate work, to be derived from the provision of

strongly-constructed and adequately-sheltered observation points for instrument-men. The Author.

The contractors' direct measurement of the crossing by means of a piano-wire in the spring of 1934 was worthy of description, as it had been a very long precise measurement. A single steel piano-wire (of 0·024 inch diameter) was stretched over a length of 2,670 feet between piers Nos. 14 and 24, those piers having been built previously. The wire was supported on wooden trestles, erected on the ice at four equidistant intermediate points. The measurement was made (after a week of waiting for a suitable calm and dull period) when the height of the tide-borne ice was such that the supports were approximately level with the end reference marks, which were established about half-way up the piers. One end of the wire was fixed, and the other, to which a weight of about 20 lbs. was attached, passed over a small pulley. Reference points were established on the wire by means of shallow saw-cuts in blobs of solder. The turning-friction of the pulley was such, however, as to render it impossible to reconcile successive readings to within less than 3 inches of each other. At the Author's suggestion, a ball-bearing bicycle wheel had been substituted for the pulley, and that expedient had been found to reduce the variation to within $\frac{1}{8}$ inch. In the subsequent precise measurement of the wire (at Lachine), the same arrangement of intermediate supports was used, and the same piece of metal was used for the weight. Temperatures were, of necessity, very carefully recorded, and the precision of the measurement was evidently within about $\frac{1}{4}$ inch.

Mr. Gill had inquired about the $1\frac{1}{2}$-inch bolts used to secure the cable-band castings to the cable. Mayari steel (so named from the locale of a Cuban deposit of ore, rich in nickel and chromium) was a chrome-nickel combination which possessed an ultimate strength of about 93,000 lbs. per square inch, and an elastic limit of about 56,000 lbs. per square inch. Great accuracy in tightening to a specified tension under field conditions was not found to be practicable, and a rough computation was made to select the length of wrench required. A 6-foot wrench was used, and there was no record of any bolt having failed in any way.

In answer to Mr. Legget's request for further information regarding the painting of the bridge, the Author observed that the exposure of the site, in so far as it affected that consideration, was not regarded as more than ordinarily severe for an inland river-location in eastern Canada. The paint used had been manufactured in accordance with the Montreal Harbour Commissioners' specifications for bridge paint to be used on the Montreal Harbour (now Jacques Cartier) bridge, that specification having been the result of co-operation

The Author. between the consulting engineers and the manufacturers. The following was a brief synopsis of the requirements of the specifications for field-coats :—

The paint was to consist of 60–65 per cent. (by weight) of pigment, and 40–35 per cent. of vehicle. The total volatile matter at 212° F. (as determined by a specified test) was not to exceed 10 per cent. in the case of the first coat, or 7 per cent. in the case of the second coat. The limiting proportions of the constituents of pigment and vehicle, expressed as percentages, were summarized below.

	First coat.	Second coat.
Pigment :		
White-lead basic carbonate	45–50	33–38
Pure zinc oxide	23–28	33–38
"Titanox"	13–18	13–18
Inorganic colouring matter and magnesium silicate	10–15	10–15
Vehicle :		
Pure linseed oil	65	75
Heat-treated china-wood oil	10	10
Driers and mineral spirits	25	15

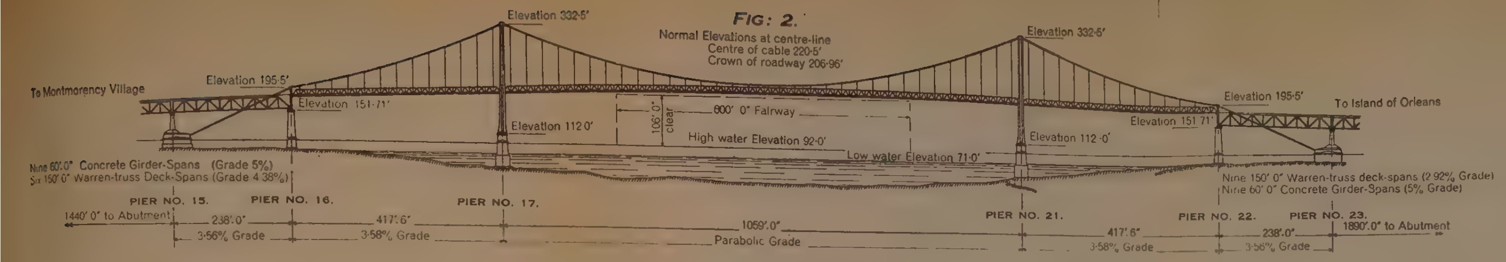
Tests relating to fineness, consistency, uniformity, and drying- and brushing-qualities were also specified, as were the methods of shipping and identification.

In conclusion, the Author wished to express his thanks to all who had contributed to the discussion, amongst whom were engineers of international renown. Their remarks and comments, he felt, formed a most valuable supplement to the Paper.

PLATE 1.
ISLAND OF ORLEANS BRIDGE.

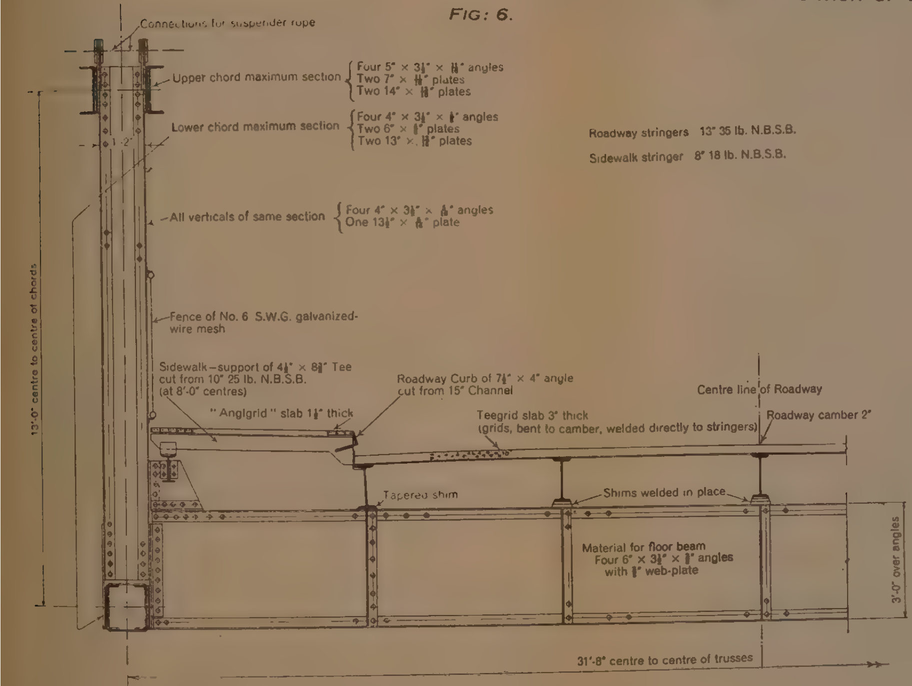
THE SUPERSTRUCTURE OF THE ISLAND OF ORLEANS SUSPENSION BRIDGE, QUEBEC.

FIG: 2.



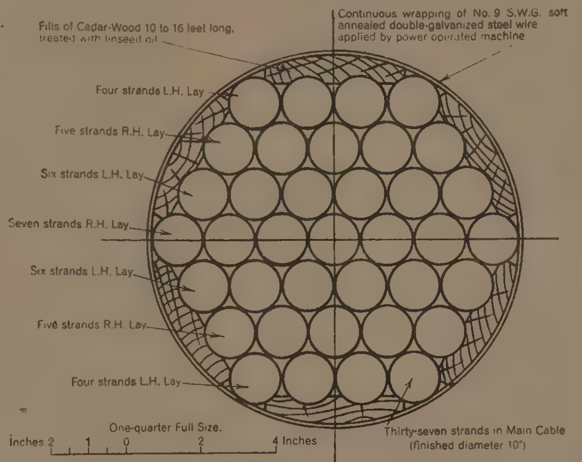
ELEVATION OF CENTRAL PORTION OF BRIDGE.

FIG: 6.



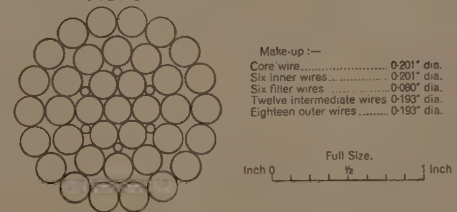
CROSS SECTION OF SUSPENDED SPAN.

FIG: 7.



CROSS SECTION OF MAIN CABLE.

FIG: 8.

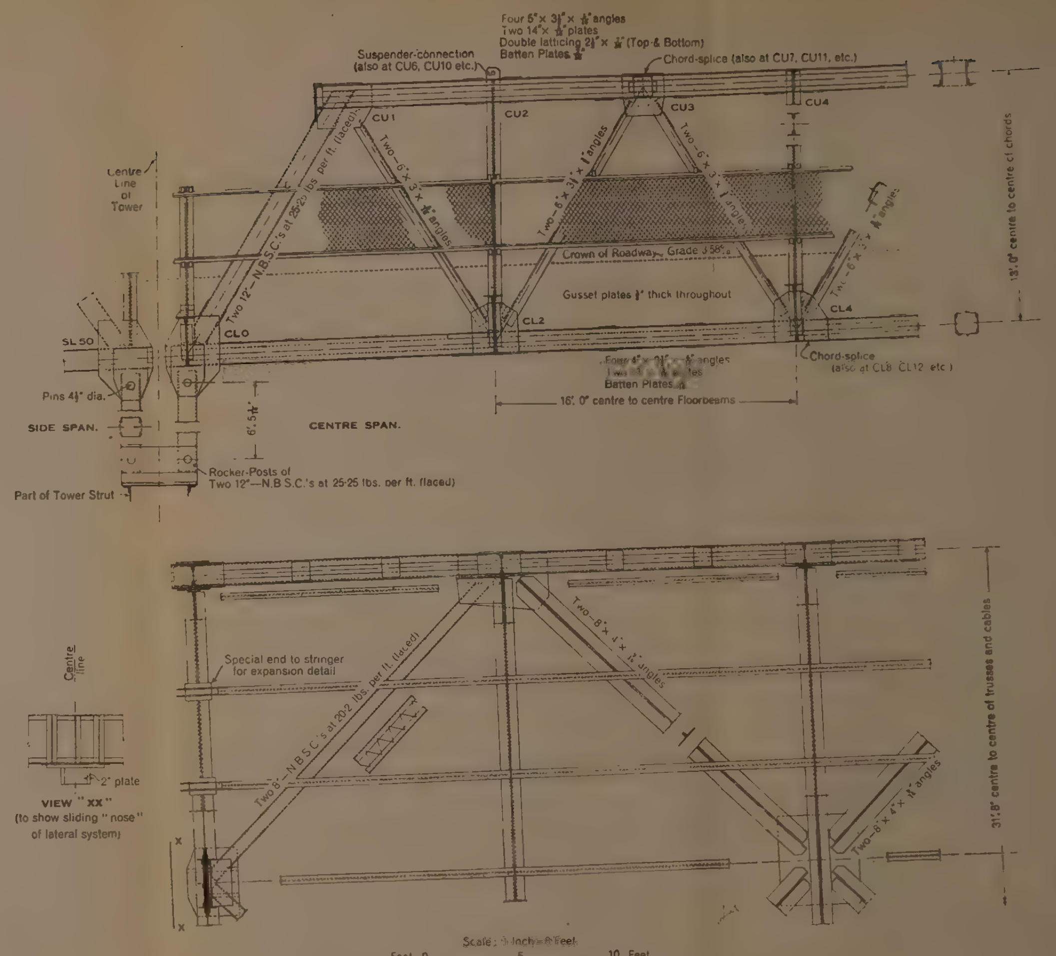


Make-up:—
Core wire.......... 0·201" dia.
Six inner wires.......... 0·201" dia.
Six filler wires.......... 0·080" dia.
Twelve intermediate wires 0·193" dia.
Eighteen outer wires.......... 0·193" dia.

Full Size.

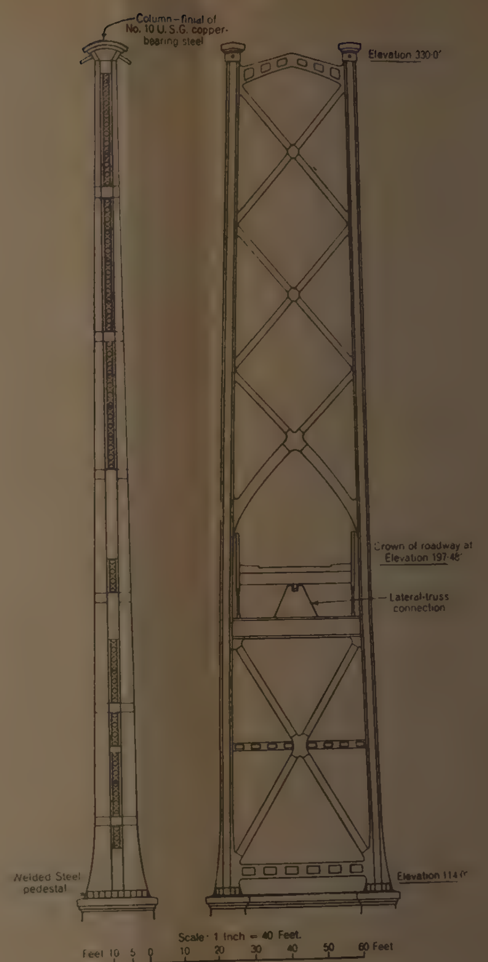
CROSS SECTION OF CABLE-STRAND.

FIGS: 5.



TYPICAL SECTION OF STIFFENING-TRUSS.

FIGS: 15.



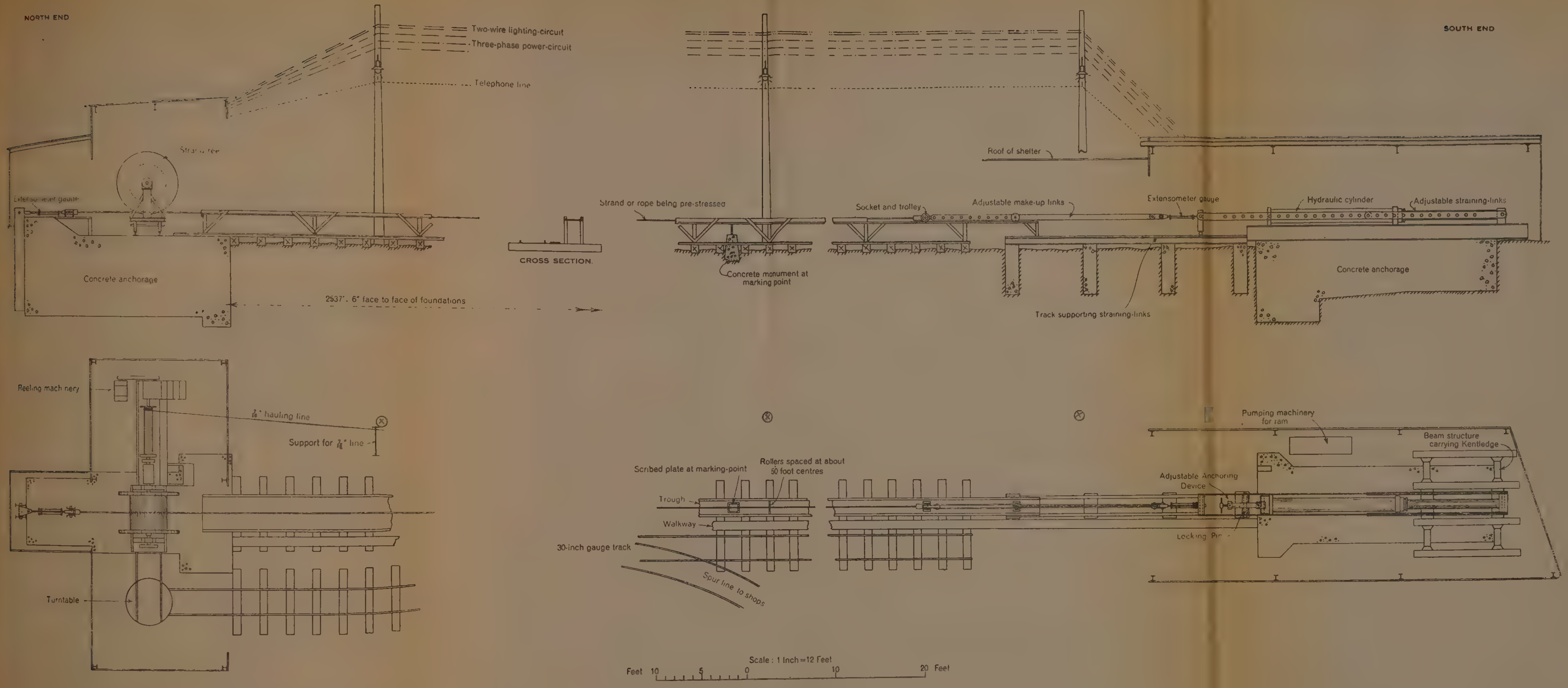
MAIN TOWER.

S. R. BANKS.

Waterlow & Sons Limited, late Thos. Kell & Son, London.

THE SUPERSTRUCTURE OF THE ISLAND OF ORLEANS SUSPENSION BRIDGE, QUEBEC.

FIGS: 33.



LAYOUT OF PRE-STRESSING PLANT.

The Institution of Civil Engineers. Journal. October, 1936.

S. R. BANKS.

WATERLOW & SONS LIMITED, LATE THOS. KELL & SON, LONDON.

ORDINARY MEETING.

28 April, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

It was resolved—That Messrs. H. R. J. Burstall, A. P. I. Cotterell, J. D. C. Couper, C. O. Grimshaw, A. J. Martin, E. H. Salmon, R. E. Tickell, and P. J. H. Unna be appointed to act as Scrutineers, in accordance with the By-laws, of the ballot for the election of the Council for the year 1936–37.

The Council reported that they had recently transferred to the class of

Members.

LEWIS RONALD EAST, M.C.E. (*Melb.*).
SAMUEL HALL.
JOHN HARLEY HARLEY-MASON.
JOHN BLACK MORRISON HAY, M.C., B.Sc. (*Glas.*), M.Sc. (*Manchester*).
EVAN BONNOR HUGH-JONES, M.C., B.Sc. (*McGill*).
EDWARD MCLAUCHLAN.
HUGH GOLD RAMSAY.

And had admitted as

Students.

RICHARD ALEXANDER ABBOT.
HUBERT ACKROYD ALLATT, M.Sc. (*Leeds*).
VIVIAN HADLEY BAYLEY.
LIONEL THOMAS CECIL BEASLEY, B.Sc. (Eng.) (*Lond.*).
WILFRED FEARN.
WILLIAM BUTLER KAVANAGH, B.Sc. (*St. Andrews*).
LESLIE DOPPING LATHAM.
EWEN GORDON MCEWEN, B.Sc. (Eng.) (*Lond.*).
ALAN FRANK MASON.
THOMAS MITCHELL, B.Sc. (*Edin.*).
FREDERICK NOEL BREWSTER PATTERSON, B.Sc. (*Durham*).
ALFRED HUGH ROBINSON.
PHILIP SHELLEY, B.Sc. (Eng.) (*Lond.*).
HAROLD STUART THOMSON.
WILLIAM WARDROP, B.Sc. (*Glas.*).
REGINALD HENRY ALBERT WEBB, B.Sc. (*S. Africa*).
ADRIAN ROBERT WILLIAMS.
KENNETH GILCHRIST YOUNG, B.Sc. (Eng.) (*Lond.*).

The Scrutineers reported that the following had been duly elected as

Associate Members.

GEORGE ROTHNEY BLAKELY, B.Sc. (Eng.) (*Lond.*) (Stud. Inst. C.E.).
RONALD BRIDGMAN (Stud. Inst. C.E.).
THORNTON HERBERT BULLOCK, B.A. (*Cantab.*).
JAMES DUNBAR (Stud. Inst. C.E.).
ROLF EDWARD GARDNER (Stud. Inst. C.E.).
LEONARD BRUCE HALEY, M.Eng. (*Sheffield*) (Stud. Inst. C.E.).
MALLINSON WALLACE HAYCOCK (Stud. Inst. C.E.).
JAMES ANDREW HENDERSON (Stud. Inst. C.E.).
HERBERT REGINALD PHILLPOTTS.
RAYMOND COLIN HASE PINCHEN, B.Sc. (Eng.) (*Lond.*), (Stud. Inst. C.E.).
VASUDEO HARI SARAPH, B.E. (*Bombay*).
PERCY SLATER (Stud. Inst. C.E.).
CHARLES WALTER STEEDMAN (Stud. Inst. C.E.).
WILLIAM KENNETH TATE, M.A. (*Cantab.*) (Stud. Inst. C.E.).
JOHN STANLEY TERRINGTON, B.Sc. (Eng.) (*Lond.*).
COLIN GEOFFREY THOMPSON (Stud. Inst. C.E.).
JOHN PERCY WALTON, B.Sc. Tech. (*Manchester*) (Stud. Inst. C.E.).

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5072.

"The Demolition of Waterloo Bridge."

By Ernest James Buckton, B.Sc. (Eng.), and
Harry John Fereday, MM. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The situation of Waterloo bridge at a bend of the river Thames in the heart of London, combined with its high level, pleasing outline and fine setting, made it one of the prominent features of London. It was one of the three great Thames bridges designed by John Rennie, the other two being Southwark bridge, completed in 1819 and replaced by a modern structure in 1921, and London bridge, completed in 1831, which still remains, having been widened in 1904. In recent years the bridge had become of considerable technical interest on account of the failure of some of its foundations and arches. Furthermore, it was obsolescent and incapable of adaptation to meet the growing demands of road and river traffic.

History.

The bridge was built by a private company for the profit to be derived from a system of tolls, and was intended to be known as "Strand" bridge. John Rennie was appointed Engineer to the

undertaking and prepared two designs, one of seven and one of nine arches, the latter (Fig. 1, Plate 1) being adopted as the less costly. The first stone was laid in October, 1811, and the bridge was opened by the Prince Regent, afterwards King George IV, on 18 June, 1817, the second anniversary of the battle of Waterloo, its title having meanwhile been changed to "Waterloo" bridge. The work was carried out by contractors, Messrs. Jolliffe & Banks, under the supervision of Rennie assisted by his son, afterwards Sir John Rennie, F.R.S., the third President of The Institution. The cost of the structure amounted to £618,370 (including parliamentary expenses) and the total cost of bridge and approaches was £937,392. It was not successful as a commercial undertaking, the existence of tolls placing it at a disadvantage since toll-free bridges existed at Westminster and Blackfriars. In 1877 it was acquired for £474,200 by the Metropolitan Board of Works under the provisions of the Metropolis Toll Bridges Act, and in 1888 it came under the control of the London County Council, as the successors to the Metropolitan Board of Works.

Between 1833 and 1877 the records of the Bridge Company refer frequently to the depositing of rubble stone around the piers rendered necessary by the serious effects of dredging and scour. In 1833 it was stated that, in consequence of dredging, the river-bed under arch No. 1 had been washed away to a depth of 18 feet below low-water level, or considerably below the foundations of the abutment, and had to be filled up with 3000 tons of Kentish ragstone. In 1847 dredging had caused scouring to a depth of 3 to 4 feet below the timber platforms at piers Nos. 2, 3, 4 and 5. In 1856 scouring, due to the wash of steamers, was reported at pier No. 2.

In 1867 pier No. 1 which, according to the records, had "always been a source of some trouble," was incorporated in the new Victoria Embankment. After the completion of the dredging operations in connection with the new Embankment no further change was noted in the bed of the river, but nevertheless the deposit of stone around the piers was continued until the bridge came under the Board of Works in 1877. In 1882 the piers were protected by the addition of aprons of concrete surrounded by timber sheet-piling.

Up to 1882 some settlement had occurred and cracks had appeared in most piers, but all the arches were sound. Local movement at pier No. 1 had been arrested by incorporation in the Victoria Embankment The settlements continued to be more or less uniform until 1923, when there were indications of further movement which developed into a serious local settlement of piers Nos. 5 and 6. In 1924 the bridge was temporarily closed to traffic, the roadway was lightened locally to reduce the load on those foundations, and

arches Nos. 5 and 6 were propped from piles driven into the river-bed (*Fig. 2*). A steel temporary bridge was constructed on the down-stream side of the old bridge and opened in September, 1925.

Since the failure of the old bridge, various proposals for dealing with it have been formulated in succession, giving rise to considerable controversy, until in 1934 a decision was reached to replace it by a structure of adequate dimensions for modern traffic. The consequent demolition of the old structure is the subject of the present Paper, and at the time of writing (March, 1936) the work is still proceeding.[1]

Construction of the Old Bridge.

From a technical point of view Rennie's structure was very interesting. Begun in 1811 and finished in 1817, its period of construction was remarkably short considering the nature of the work, the difficulties of the site and the lack of mechanical appliances. The foundations, as was then customary, consisted of timber platforms supported on closely-spaced timber piles (*Figs. 3*). There were nearly three thousand piles in the work, of lengths approaching 20 feet. The timber platforms were set at levels ranging from about low-water level in the case of the abutments to about 7 feet below low-water level in the case of the deepest of the piers.[2] These foundations were carried out within large timber clay-filled cofferdams, oval in plan. Without modern materials for making cofferdams and without modern mechanical plant for pile-driving and pumping, not to mention quarrying, transport, and other work, it was a wonderful feat of organization to put up such a structure, containing about 100,000 tons of dressed masonry, in so short a period.

Main Features.

The structure consisted of nine arches (*Figs. 3*, p. 475) having elliptical soffits of 120 feet clear span and 35 feet rise, eight piers each 20 feet thick, and two massive abutments with wing-walls and staircases leading down to the foreshore. The bridge was level throughout, and provided a carriageway of 27 feet 6 inches and two footways each 7 feet 6 inches wide. The arch springings were set at about mean tide level, and soffit-level at the crown was about 26½ feet above Trinity High Water. The voussoirs and face-stones were chamfered, and other features included two Doric columns on each of the pier-

---

[1] All arches have been removed, and cofferdams for the removal of piers Nos. 5 and 6 completed. The whole of the masonry and the timber platform to pier No. 5 has been removed and the removal of pier No. 6 is in progress.

[2] Old London Bridge was still in existence at this time, and, owing to the obstruction that it caused to the flow of the river, low-water level was higher by some feet than it is now.

Fig. 2.

ARCH No. 5 AS PROPPED IN 1924.

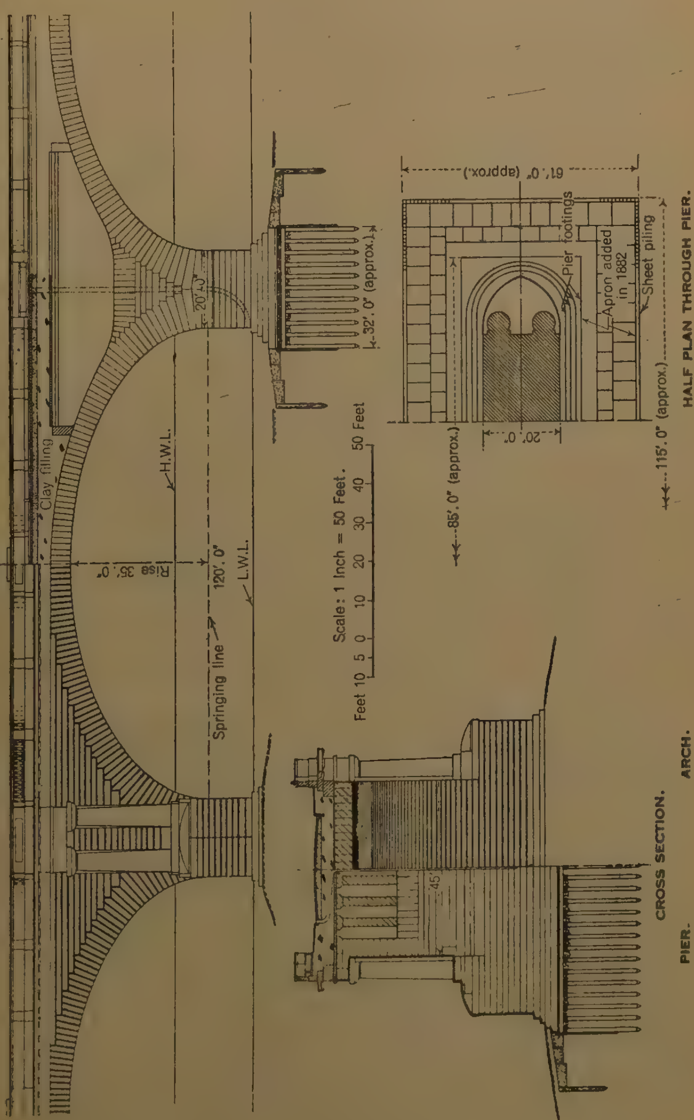
5072.

Fig. 5.

DEMOLITION SCHEME: REMOVAL OF ARCHES.

faces, a moulded cornice having a wide overhang, and an ornamental balustrade with embayments over each pier. In 1908 the north-western stairway was removed to make way for the entrance to the Kingsway tram-subway.

Figs. 3.



TYPICAL PIER AND ARCH.

Design.

The arch-stones were nearly 5 feet deep at the crowns, increasing uniformly to about 9 feet deep at the haunches (*Figs. 3*). Over the crowns there was a depth of about 4 feet of clay filling below the road-material, increasing to a depth of about 9 feet at the quarter-points, where it was retained by transverse head-walls. Over the spandrels and the piers, weight was somewhat reduced by the substitution of longitudinal spandrel-walls, consisting of two outer walls 5 feet thick of granite and sandstone, and six interior brick walls 2¼ feet thick having 3-foot spaces between them, which were covered by York stone slabs resting on stone corbels. Above the slabs there was a 3½-foot thickness of clay filling beneath the roadway-material. The stone courses in the arches were all set radially until they merged into the horizontal courses of the piers. The arches were correct in design in that throughout all stages in loading from that of the arch-stones only (which may have occurred during construction when the centering was stripped) to "full load," the line arch was well within the middle third of the arch-rings, the only point of deviation from the ideal being that the direction of the joints adjacent to the piers was not normal to the direction of thrust, as is inevitable in a masonry arch having an elliptical soffit. The obliquity caused a shear along the joint in an upward direction which would be resisted by friction and the strength of the mortar as well as by contact of the arch-stones with the spandrel-walls.

It is interesting to note that in every arch, at four of the joints between courses of voussoirs, wrought-iron flat bars, 4 inches by 1 inch in section, were built in for nearly the full width of the arch, sunk into a chase in the stone and run in with lead. There was remarkably little corrosion in this ironwork.

The piers (*Figs. 3*) were 45 feet by 20 feet in plan above the cut-waters, and thus had a net area of 900 square feet, exclusive of the two ornamental columns at each end. The timber raft on which each pier was founded was approximately 85 feet by 32 feet, thus having an area of 2,720 square feet. The area of a pier and cutwaters was about 1,300 square feet, and was thus only half the area of the raft; the increase of 100 per cent. in area was effected in a total height of footings of only 6 feet. The result has been that the footings, particularly round the cutwaters, have broken off and become steeply inclined to the horizontal. The gross weight of a pier, including cutwaters, footings, and superstructure, was about 10½ thousand tons, or about 9½ thousand tons excluding cutwaters. It is difficult to estimate the actual loads per square foot on the foundations, and the only practical approximation is, firstly, to consider the whole load over the whole area, and, secondly, to assume that the

cutwaters which had broken off from the piers carried no other load than their own weight, and that only half of the area of the footings which had turned up was effective. The loads on the foundations calculated under these two assumptions are given in Table I.

TABLE I.

Assumption.	Load: tons.	Bearing area: square feet.	Load: tons per square foot.	Load: tons per pile.[1]
Whole of footings effective .	10,500	2,720	3·86	48
Cutwaters excluded, half of footings effective . . .	9,500	1,125	8·45	104

[1] The loads per pile have been computed on the assumption of 220 piles to the gross area; the actual number of piles under each pier varies, according to available information, from a minimum of 176 to a maximum of 319.

Although the cutwaters cracked off from the piers, they did not either appreciably tilt or change their level relative to the piers; therefore, in estimating foundation-stresses and pile-loads a possibly more accurate and less severe approximation would be to add to the area of the pier and cutwater (1,300 square feet) half the area of the pier-footings over the length of the pier (45 feet by 5 feet, or 225 square feet). The total bearing area would then be 1,525 square feet, the load per square foot would become slightly less than 7 tons, and the load per pile over 80 tons. It will be seen from the preceding calculations that the actual loads per square foot or per pile were extremely high.

The original design is therefore open to adverse criticism under two heads:—

(*a*) On account of the great spread given to the footings without sufficient provision for ensuring a corresponding spread of the load.

(*b*) Because by simply raising the level of the spandrel-walls and stone slabs and modifying them, 1,000 tons or more in weight of clay filling, etc., might have been eliminated from every span and the foundation-loads thus materially reduced. That Rennie realized later the defects in the design of the footings is evidenced at London bridge, designed by him and built between 1824 and 1831 under the supervision of his son. A similar spread of footings is there distributed in a height of 20 feet, instead of 6 feet as at Waterloo bridge.

Some of the timber rafts were founded on a considerable thickness of ballast and some were taken down almost to the clay, which varies considerably in level at different piers. The piles in all cases were driven well into the clay.

Workmanship and Materials.

The workmanship was extremely good throughout. The perfection of the work in the exterior features, such as the balusters, coping, and cornice, has been apparent to all Londoners for a century, but it was only in the demolition of the bridge that it became known how perfectly every arch-stone had been cut to taper and how solid and faultless had been the interior construction of the piers. There were practically no voids. The filling of the observation holes in the piers with liquid grout, as explained later, resulted in hardly any loss of this material. The lime mortar in which the masonry was built was exceedingly good.

The materials used were, for the parapets, Cornish granite with Aberdeen granite for the balusters ; for the piers, arches and external spandrel walls, Cornish granite mixed with perhaps 20 per cent. of sandstone. The internal spandrel-walls were of hard strong brick of good quality.

The rafts and piles were of timber in which fir, elm and beech predominated.

Action taken by London County Council.

In December, 1923, remedial measures were taken to deal with the settlement which had occurred at pier No. 5 and the distortion in the adjacent arches.

In April, 1924, the London County Council decided to reconstruct and widen the bridge, while preserving in the new bridge the character and identity of the existing structure.

In July, 1924, the London County Council, having rescinded its resolution of April, 1924, appointed a Special Committee on Thames Bridges, with instructions to consider and report *inter alia* on the reconstruction of Waterloo bridge from the point of view of both road and river traffic.

In December, 1924, the Council of The Institution were asked for their views on the question as to whether it would be practicable and reasonable to underpin all or some of the piers in order to render the structure safe and to enable it to be restored to its original form. They replied that in their view the London County Council would be well advised to act on the considered advice of their consultants, Mr. (now Sir) Basil Mott, and the late Sir Maurice Fitzmaurice, Past-Presidents Inst. C.E., who had reported unfavourably as to underpinning.

In February, 1925, the Special Committee on Thames Bridges reported, and the London County Council expressed its opinion that it was desirable, subject to the provision of a subway underneath

the Strand for the accommodation of the general vehicular traffic using the bridge, to reconstruct Waterloo bridge with not more than five river-arches and of width sufficient for six lines of vehicular traffic.

Following the issue, in June, 1925, of a "Report of the Conference of Societies urging the Preservation of Waterloo Bridge" the whole matter was reviewed.

In December, 1925, the London County Council rescinded its resolution of February, 1925, with its reservation as to the provision of a subway under the Strand, and decided to take steps to construct a new six-line bridge with not more than five arches over the river, but in June, 1926, action was deferred in consequence of the appointment of the Royal Commission on Cross River Traffic in London. Following the issue in November, 1926, of the report of the Commission, it was decided in March, 1927, to recondition the bridge, as recommended by the Commission, provided that steps were taken to construct a new bridge at Charing Cross. The rejection of the London County Council (Charing Cross Bridge) Bill, 1930, was followed by the investigation by the Charing Cross Bridge Scheme Advisory Committee, under the chairmanship of the Right Hon. Sir Leslie (now Lord Justice) Scott, and the decision of the Council to seek legislation again for a Charing Cross bridge. His Majesty's Government decided that it was not possible at that time to renew the offer of a 75-per cent. grant from the Road Fund, and the proposal was not proceeded with. In February, 1932, the London County Council again decided to construct a new and widened Waterloo bridge, on the understanding that a grant of 60 per cent. of the cost would be made from the Road Fund. The necessary financial provision was, however, deleted from the Council's annual Money Bill of 1932 consequent on an instruction to that effect which was carried by the House of Commons.

Following discussions between representatives of H.M. Government and the London County Council, on the instructions of the Council a report was prepared in October, 1932, by the late Sir Frederick Palmer, Past-President Inst. C.E., of the Authors' firm, with estimates for

(*a*) reconditioning the bridge, together with such widening, by corbelling, as would enable the bridge to carry four lines of traffic;
(*b*) reconditioning the bridge within its existing dimensions; and
(*c*) replacing the piers and arches which were in the worst condition by steel girders and other temporary supports, the existing stones being stored for future use if required.

The Minister of Transport notified the Council in January, 1933,

that the Government were prepared to make a grant from the Road Fund of 60 per cent. of the cost of reconditioning the bridge as proposed in Sir Frederick Palmer's report. The London County Council thereupon "reluctantly" agreed to recondition the bridge and widen it, by corbelling, to take four lines of vehicular traffic, and the Authors' firm was instructed to prepare the necessary drawings and documents and to call for tenders, which were duly obtained. However, before any tender was accepted, the newly-elected Council decided in June, 1934, to demolish the bridge and to build a new one of sufficient width to accommodate six lines of vehicular traffic.

Schemes for Reconditioning by Underpinning.

Sir Frederick Palmer had been instructed in March, 1933, to prepare a reconditioning scheme and documents for tendering. An underpinning scheme was devised and tenders invited in February, 1934, freedom being given to tenderers to submit alternatives to the formulated scheme if they wished. Piers Nos. 2, 3, 4 and 7 were to be underpinned, and arches Nos. 5, 6 and 7, with piers Nos. 5 and 6, were to be taken down and rebuilt. A new reinforced-concrete deck was to be provided and an increased width of roadway obtained by corbelling.

The underpinning, if it had been adopted, would probably have proved more interesting than the present subject, but it cannot be dealt with in this Paper, being only a proposal not carried into execution.

CONDITION OF BRIDGE PRIOR TO DEMOLITION.

TABLE II.—TOTAL SETTLEMENT OF PIERS SINCE 1820: INCHES.

Date.	Pier Number.							
	1	2	3	4	5	6	7	8
1881	4·7	0·6	0·6	1·9	2·1	1·6	0·7	0
1901	4·9	2·7	1·8	3·5	5·5	5·0	2·3	0·6
1924	5·1	4·1	3·7	6·4	28·4	10·1	2·9	0·6
1931	5·4	5·5	5·1	7·0	28·9	13·0	3·2	0·6
1934	5·4	6·4	6·1	7·3	29·1	13·8	3·2	0·6

Table II shows that piers Nos. 1, 2, 3 and 4 had each settled about ½ foot and pier No. 7 about ¼ foot in a period of 115 years—perhaps

remarkably small settlements considering the intensity of the loads that they carried and the imperfect design of the footings. Such settlements, although doubtless ominous, were in themselves of little consequence, but what was serious was that pier No. 5 had settled over $1\frac{3}{4}$ foot relative to pier No. 4 and $1\frac{1}{4}$ foot relative to pier No. 6. It was this relative settlement of adjacent piers that caused arches Nos. 5 and 6 to distort, with results so alarming that in 1924 the bridge was closed to traffic for a period during which lightening in weight of the superstructure was effected over pier No. 5, and arch No. 5 and a portion of arch No. 6 were propped. As further settlement of pier No. 5 seemed at that time to be likely, these precautionary measures were justified, but it is doubtful whether the propping ever carried any considerable load, and in fact the pier ceased to settle before these measures were completed.

The settlement of pier No. 5 relative to piers Nos. 4 and 6 affected arches Nos. 5 and 6, as shown in Fig. 4, Plate 1, with the result that the distance between the adjacent sides of these two arches across the top of pier No. 5 became reduced. This caused the top square-cut corners of the arch-voussoirs to "mesh" with the horizontal stone courses in the external spandrel-walls. The resulting pressure was very great, causing the spalling-off of some of the stone surfaces in contact. The effect in the structure was that a relieving arch became induced from pier No. 4 to pier No. 6 in the voussoirs of arches Nos. 5 and 6 adjacent to piers Nos. 4 and 6 and by strut-action along the external spandrel-walls across the top of pier No. 5.

Demolition Scheme.

The principles underlying any scheme for the demolition of the bridge would include such methods of support as :—

(*a*) Propping.
(*b*) Arched centering.
(*c*) Suspended centering.

A scheme consisting of propping only was not wholly practicable, because the Embankment arch had to be kept open to vehicular traffic, and arches Nos. 3 and 4 for river-traffic.

Since soffit-level of the arches of Waterloo bridge was 7 feet higher than that of Westminster bridge, a method of demolition by means of propping in the south half of the bridge, and of steel arches fixed under the existing arches in the north half of the bridge, was possible. The Authors were inclined to favour this scheme when they first considered the problem some years ago, but it had the disadvantage that the navigation-space below the masonry arches would be more

curtailed by steel arches than by suspended centering. Methods of demolition by the use of suspended centering had previously been considered by both Sir George Humphreys, Past-President Inst. C.E., and Sir Basil Mott. A new scheme on these lines was devised by the London County Council's engineers and handed over to the Authors in June, 1934. On examining it, the Authors considered it to be a good workable scheme, and they at once adopted it in principle and set to work to develop it in detail (*Fig. 5*, facing p. 475, and Fig. 6, Plate 1).

The scheme consisted in first lightening the bridge by the removal of the superstructure above the level of the top of the arch key-stones, then erecting four lines of girders bearing on the piers and spanning over the arches. Suspender-rods from these girders were to pass through holes drilled in the masonry and to support steel centering erected from below. The arches were then to be demolished while remaining self-supporting until about only a quarter of the original width of 45 feet remained in the middle. The weight of this middle strip was then to be transferred to the girder system above, the arches broken and the stone removed. All arches except the propped arch No. 5 were to be demolished in this manner. When all the arches had been removed, including arch No. 5, which was to be demolished on its propping, the girders were to be removed by "cantilevering back" and the piers and their foundations removed two at a time within steel sheet-piled cofferdams driven round them. In this manner the work could be carried out in safety and without causing undue inconvenience to river-traffic.

Contract.

A contract on a "value—cost" basis for the demolition of the old bridge and the maintenance of the temporary bridge was placed in June, 1934, with Sir William Arrol & Company, Ltd., who had constructed the temporary bridge in 1924–25 and whose services had been retained since then in connection with the maintenance both of the temporary bridge and the propping of arches Nos. 5 and 6 of the old bridge.

Preparation of Piers and Abutments to support the Gantry-Girders and for Subsequent Demolition.

General.

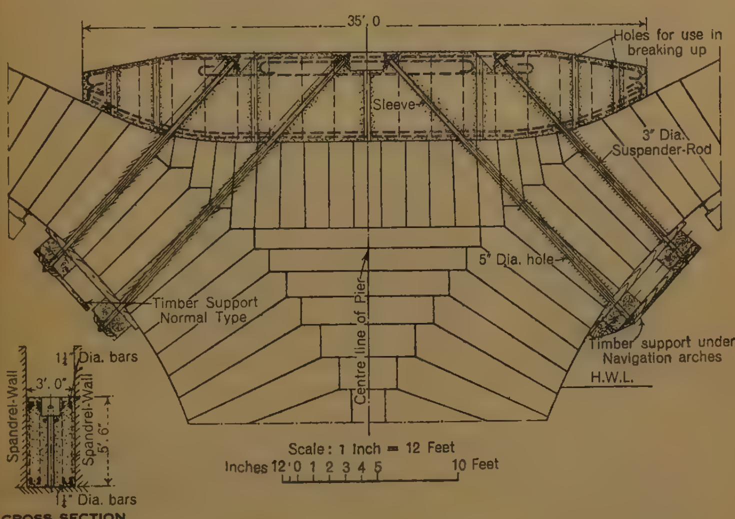
The weight of the gantry-girders and suspended centering was about 300 tons per span, and the weight of the central portion of the arch that the two inner gantry-girders had subsequently to support was about 700 tons. Thus each pier had to be adapted

so as to carry with safety, when deprived of the lateral support of the arch-thrusts, a vertical load of 1,000 tons placed centrally on it, and the abutments had to be rendered safe under similar conditions of loading. Similarly, it was necessary in each case to prevent the remaining inclined arch-stones from sliding off.

Piers.

The arrangement of the stonework in the piers was such that they would not with safety carry vertical loads when deprived of the lateral support of the arches. Sixteen diagonal holes were therefore

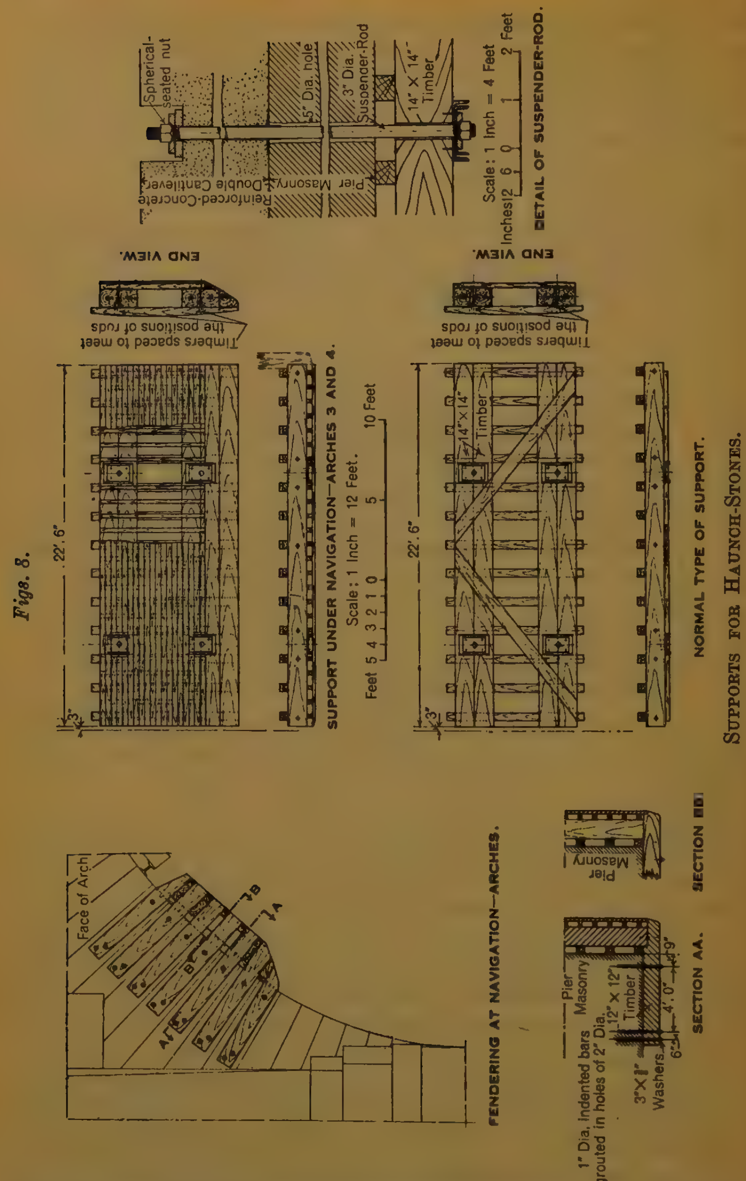
Figs. 7.



SUPPORTS FOR HAUNCH-STONES.

drilled in each pier, through which 3-inch diameter rods were passed in such a configuration that these rods were held at their tops in groups of four longitudinally with the bridge by tightening them up against strong reinforced-concrete double-cantilever beams specially constructed in the space between the spandrel-walls (*Figs.* 7). At their lower ends these bolts, in groups of four, engaged timber "supports" (*Figs.* 8, p. 484) for the retention of the arch voussoir-stones that remained attached to the piers after the intervening portions of the arches had been demolished. These diagonal bolts were kept screwed up very tightly.

The gantry-girders took their bearing on new concrete bedstones lightly reinforced, constructed on the old spandrel-walls. Before,



Figs. 8.

SUPPORTS FOR HAUNCH-STONES.

however, these girders were subjected to loads, other than their own weight and that of the gantry cranes, the construction was stiffened by means of new brindle-brick piers built in the spaces between the old spandrel-walls. Such a construction, although only for

temporary use, required a pier having a solid hearting on which to rest. In order to test the solidity of the piers, three 5-inch diameter observation-holes were drilled in each to a depth of 20 feet below the pier top, that is, right down to the horizontal courses. An electric lamp attached to a standard having a mirror set at 45 degrees was lowered into the holes and enabled an observer at the top to see the joints in the masonry pierced by them. No bad joints were observed, and on grouting the holes very little grout was lost.

Further points in the design of these double cantilevers and diagonally-suspended "supports" were :—

(*a*) The length of the beams was sufficient to cover the tops of the voussoir-joints that were seriously inclined to the line of arch-thrust, so that no upward slip could occur in the process of the removal of arch spandrel-walls.

(*b*) By this arrangement a very heavy load of voussoirs was kept in contact with the pier on each side, the resulting inclined inward reaction compensating to some extent for the lost lateral thrust from the arches.

(*c*) Any interior arching effect that might develop in the hearting of the pier would have the effect of tending to push out laterally the voussoir-stones below those directly retained by the supports. Such effect would be resisted by double friction on these voussoirs and by the taper on them which would, if movement occurred, raise bodily the group of voussoirs retained by the support, thus raising the cantilever-beams and all loading above.

(*d*) There was a large reserve in longitudinal strength, both in the diagonal bolts themselves and in the reinforcement of the double cantilevers.

(*e*) Longitudinal strength in the supports was provided by twin 14-inch by 14-inch timbers to which lagging was attached. The arrangement was thus flexible, as the twin timbers could be inclined to meet the actual position of the bolts even when the bolt-ends were seriously out of their nominal positions owing to errors in direction in the drilling from the top of the long diagonal bolt-holes through which they passed.

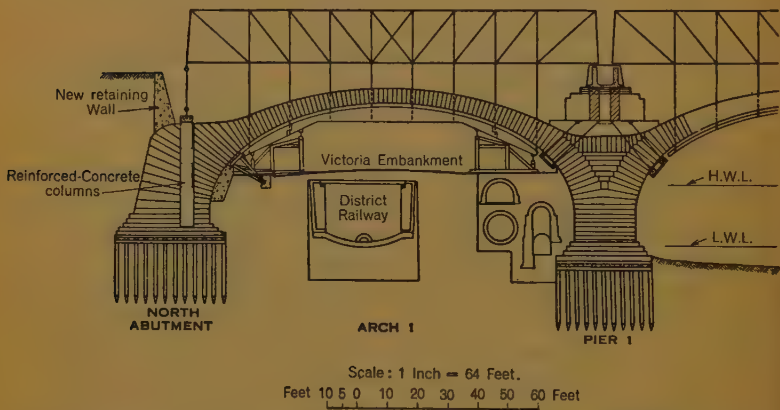
(*f*) In the case of the navigation-arches, these timber supports were strongly boxed in on their soffits to resist abrasion and to protect the bolt-heads. The ends were protected by short lengths of whole timbers bolted on to the face of the supported voussoirs.

North Abutment.

The abutments at each end are composed of masonry courses laid to steep slopes so as to be approximately normal to the line of thrust. The abutments also perform the function of retaining walls subject to both overturning and forward movement. Both abutments had therefore to be adapted to stand as retaining walls and to support the vertical loads imposed in the course of demolition when deprived of the lateral thrust of their arches.

On the north side a new retaining wall had to be built on the abutment (*Fig. 9*), located behind the old returned ends of the spandrel-walls which were demolished to make way for new double-pinned

Fig. 9.



steel columns to the gantry-girders. At the soffit water had found its way down the arch-stones and had caused mud to form between the under surface of the arch-stones and the soil below the paving on the Embankment. Therefore, to prevent forward movement and tilting, a trench was excavated and filled with concrete between the soffit and soil under the Embankment. For further security, the trench was tightened by jacking as it was timbered.

In order to take the loads from the girders down to horizontal courses and to take up the horizontal shear set up by the sloping courses and retaining wall founded on them, four holes $4\frac{1}{2}$ feet square and 34 feet deep were excavated, one at a time, in the masonry. The sides of these holes were then rendered and covered with two layers of felt, and reinforced columns were constructed within, having certain lengths of their sides and front not in contact with the masonry. These columns were thus enabled to shorten elastically under their loads and to act as cantilevers. At the same time, the

arch-thrust that then existed could pass through them. Sliding of the top sloping courses was prevented by propping.

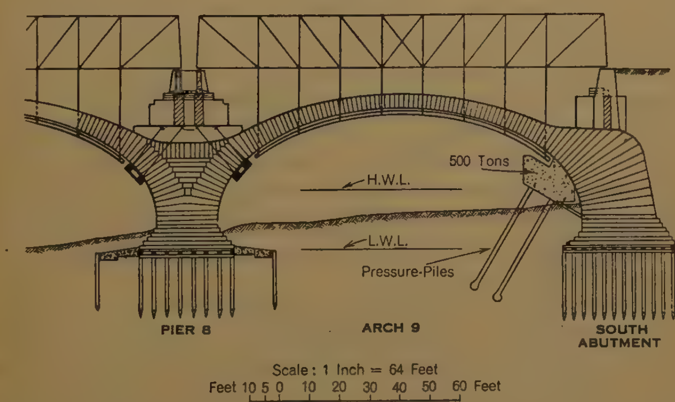
Longitudinal forces under the girder-bearings were eliminated by placing a system of double-pinned steel columns between the lintel over the reinforced-concrete columns and the girder-bearings.

South Abutment.

As there was no embankment in front of the abutment on the south side, the problem of making the necessary provision for its security was different from that on the north side.

Girder bedstones in concrete, lightly reinforced, were constructed over the old return end-walls of the original spandrel-walls (*Fig. 10*). These vertical loads thus became supported directly off the sloping

Fig. 10.



courses of the abutment. In addition, the horizontal and vertical forces due to the earth-pressure behind came on to these courses. As a result of calculation, making allowances for friction and other uncertainties, a resultant force estimated at 700 tons in the direction of the sloping courses at 45 degrees to the horizontal had to be resisted. This was done by forming thirty pressure-piles in the soil in front of the abutment, raked at 30 degrees to the vertical. Each pile was separately tested to a load of 60 tons and was incorporated in a light reinforced-concrete cap on which a load of 500 tons of concrete was placed in contact with the abutment so as to deflect the estimated force acting in a direction of 45 degrees to the vertical to an inclination equal to that of the piles. Small reinforced-concrete regulators were also built in below, which, by acting as struts or ties, would tend to keep the resultant force to the inclination of the piles

either if the estimated figure of 700 tons were exceeded or if it were not wholly attained.

Details of Demolition Scheme.

Steelwork.

The first necessity was to lighten the whole of the structure by taking away the parapets, pavements, road-surfacing materials, cornice and frieze course and the large quantity of clay filling covering both the arch-stones and the stone slabs over the hollow spandrels. By this means approximately 2,000 tons of material was removed per span, leaving a level deck of the full width of the bridge. The only precautions necessary in this part of the work were those of keeping the loads on adjacent spans approximately equal, and of arranging the special hump-loading over the distorted arches Nos. 5 and 6. The steelwork was erected on the deck thus provided.

The steelwork consisted of two inner and two outer girders, set at 12 feet centres, at all spans except arch No. 5, which was propped. Each girder was of a truss type having eight panel-points from which suspender-rods were hung. The suspender-rods from the outer girders were of 2½ inches in diameter, and those from the inner girders were 3 inches in diameter for the six central rods and 3½ inches in diameter for the end rods. The suspender-rods had to pass through the arches, thus necessitating the drilling of thirty-two vertical suspender-rod holes per span.

The steelwork in the girders was fabricated at Messrs. Arrol's works at Glasgow, sent to the work by sea and delivered at the site in barges in pieces not exceeding 5 tons in weight. These pieces were lifted from the barges by the derrick-cranes on either side of the propped arch 5 and stacked on this arch, from which they were fed first to the adjacent spans for erection. The girder-work was built up off timber packings at about 12 feet spacing. As soon as a pair of inner gantry-girders was completed, material for the next span was transported along their length on bogies running on a 2-foot-gauge track between them.

The gantry-girders were designed to clear the arches and to span from pier to pier, and, by means of their thirty-two suspender rods passing in the holes drilled through the arches, to carry the centering. The two outer girders were designed to support only occasional local loads consisting of a few arch-stones; the inner girders, however, were designed to carry the full weight of a 12-foot width of the central strip of the arches.

The suspended centering to each of the river-arches was in seven separate portions, connected by articulated joints so as to ensure

close contact with the arch soffit. The centering of the Embankment arch was also suspended, but in a rigid length of 68 feet over the central part, with two articulated ends each 15 feet in length.

Each section of the centering to the river-arches was erected on a barge and mounted on lifting girders. The barge was towed to a position immediately below that to which the section belonged. A specially-designed frame was placed on the top of the gantry-girders above, from which wire-rope tackles were hung and shackled to the lifting girders. The sections were raised by means of four 3-ton hand winches mounted on the frame.

The time taken for the erection of each section, inclusive of the towing away of the barge and its return to its berth in the centre of the river, was $1\frac{1}{2}$ hours.

The Embankment arch was specially surveyed so as to get the centering fabricated to fit with as little clearance as possible. This centering contained four lines of girders, which were to be placed under the four gantry-girders and suspended from them, and two fascia girders. Each of these six girders contained a central portion, capable of spanning 56 feet clear between its erection supports, and two articulated end-pieces. The six central portions were each delivered in two pieces which were connected up at site and lifted by the two travelling gantry-cranes into position at the west end of the Embankment arch, whence they were hauled in under it on sliding ways, after lagging-joists and skin-plating had been placed and welded.

After the centering under the Embankment arch had been drawn up close under it by means of turnbuckles on the suspender-rods, all voids between the skin-plating and the arch-stones were filled with dry sand poured through the holes for the suspender-rods and blown in from the sides by compressed air. When the arch-stones were removed it was found that this method of filling the voids had been effective.

Preparation of Arches Nos. 5 and 6 for Demolition.

Since pier No. 5 had settled so considerably relative to piers Nos. 4 and 6 adjacent to it, careful attention was necessary in preparing for the demolition of arches Nos. 5 and 6, particularly as it was decided to remove arch No. 6 in the same manner as all the other arches (except arch No. 5) by means of steel centering slung from overhead gantry-girders. Thus it became necessary to remove the propping under arch No. 6. This had been in place for 10 years, and the effects of its removal on both arch No. 6 and the sunken pier No. 5, which would then become propped on one side only, demanded consideration. Both arches Nos. 5 and 6 were distorted badly, and

the effectiveness of the propping was uncertain. Experiment on a model, as well as observation at site, gave clear evidence of the existence of arch-action in the spandrel-walls over pier No. 5.

It was obvious that every precaution should be taken to prevent these two arches from changing their shape. The joints in the soffits of both arches were carefully inspected and found in many cases to be open, but there was no evidence of the existence of a dangerous horizontal arch across the width of either arch due to the pressure on the external spandrel-walls. This was a possibility that had to be considered, because no arch-action of any magnitude could take place across the top of pier No. 5 in the material of the internal brick spandrel-walls for two reasons, firstly, because there were no square-cut joints to " mesh " and, secondly, because inspection of the pressure at the " cogs " in the external spandrel-walls revealed that brickwork would under similar conditions crush locally and thus fail to transmit the pressure. Before proceeding, therefore, to eliminate arch-action on the external spandrel-walls, the following precautionary measures were adopted :—

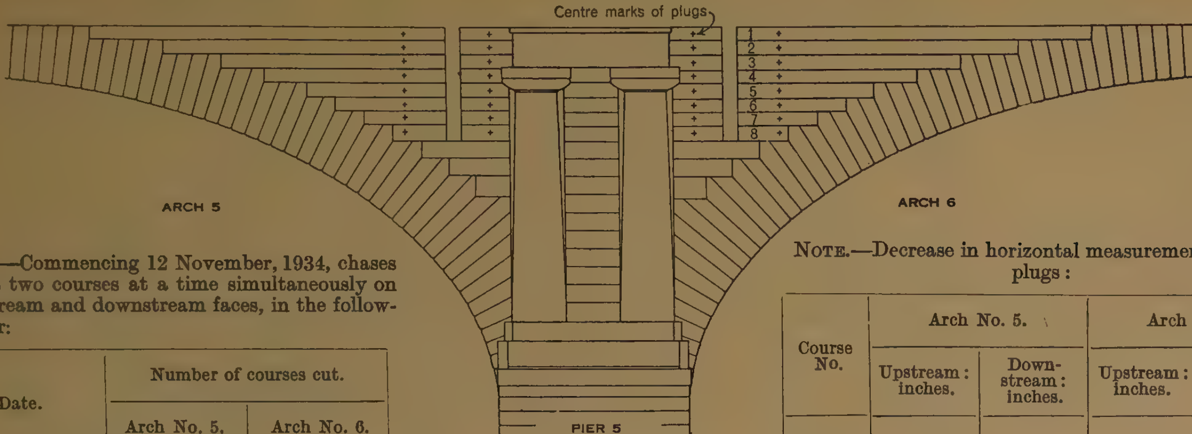
(*a*) All open joints between the voussoir-courses were caulked throughout arches Nos. 5 and 6. This caulking was carried out in lead, which proved to be the most suitable material for ramming up from below. Inspection was made of the extrados-courses by entering the vaults, but no voids in the joints on the extrados of these arches were found.

(*b*) Both these arches were hump-loaded on their half-spans remote from pier No. 5, in order to induce a " line " arch in them in conformity with their deformed shape.

(*c*) The propping to both arches was inspected and was generally found to be very lightly loaded. The wedging of the propping to arch No. 6 was kept hand-tight only, and that to arch No. 5 was driven tight.

Four vertical chases were cut, two through the upstream and two through the downstream spandrel-walls, one on either side of pier No. 5, between vertical lines inscribed on previously-placed brass plugs (*Fig. 11*). The resulting reduction in the horizontal distance between these lines was considerable and the soffit-level under the crown of arch No. 6 was lowered appreciably. The chases were necessary on both sides of pier No. 5 because a chase on one side only would not have eliminated all strut-action, since it would have continued to exist in the uncut spandrel, being maintained by friction across the horizontal courses and so down to the pier. It had always been evident, by inspection, that the force was greater in

Fig. 11.



NOTE.—Commencing 12 November, 1934, chases were cut two courses at a time simultaneously on the upstream and downstream faces, in the following order:

Date.	Number of courses cut.	
	Arch No. 5.	Arch No. 6.
22.11.34	0	2
29.11.34	0	4
5.12.34	0	6
12.12.34	2	8
17.12 34	4	8
19.12.34	6	8
29.12.34	8	8

NOTE.—Decrease in horizontal measurement between plugs:

Course No.	Arch No. 5.		Arch No. 6.	
	Upstream: inches.	Down-stream: inches.	Upstream: inches.	Down-stream: inches.
1	$\frac{1}{2}$	$\frac{13}{32}$	$\frac{1}{16}$	$\frac{1}{8}$
2	$\frac{11}{32}$	$\frac{5}{16}$	$\frac{1}{8}$	$\frac{5}{32}$
3	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{3}{16}$
4	$\frac{1}{8}$	$\frac{1}{8}$	0	$\frac{1}{16}$
5	$\frac{1}{16}$	$\frac{3}{16}$	$\frac{1}{16}$	$\frac{1}{16}$
6	$\frac{1}{16}$	$\frac{1}{16}$	0	$\frac{1}{32}$
7	$\frac{1}{32}$	$\frac{1}{32}$	0	$\frac{1}{32}$
8	$\frac{3}{32}$	$\frac{1}{16}$	0	$\frac{1}{16}$

VERTICAL CHASES IN ARCHES NOS. 5 AND 6.

the spandrel-walls in contact with arch No. 5 than in those in contact with arch No. 6.

When the cutting of all chases was completed, it was proved by observation of levels and examination of the propping that arch No. 6 was then acting as a true arch and that arch No. 5 was acting correspondingly as a bent strut. The wedges under arch No. 6 were then further slackened and the propping subsequently removed. By these means all risk of any dynamic effect in the process of the re-distribution of the forces in these arches was eliminated.

Procedure in the Demolition of the Arches.

Throughout the period of demolition of the arches, arch No. 5 was always kept more heavily loaded than the arches adjacent to it so that it could always resist any tendency for piers Nos. 4 and 5 to rotate towards it, while the props under it would always prevent the reverse tendency. Arch No. 5 and, to a lesser extent, arch No. 6 were always kept hump-loaded on their high sides (those remote from pier No. 5) in order to induce lines of pressure in conformity with their distorted shapes. A load was also kept during certain stages on arch No. 7 in order to reduce the inequality of loading on the two sides of pier No. 6.

After the preparatory work described, the remaining clay filling was removed at all arches more or less simultaneously, followed by the simultaneous and symmetrical removal of all spandrel-walls.

The actual removal of the arches themselves was commenced (Figs. 12, Plate 1) by "nibbling," that is, taking away from all arches in succession a few voussoirs at the crowns on both up- and down-stream sides. This was carried on until the 12-foot strip was left in the centre and the arch-stones in the "outer" strips were racked back at about 45 degrees in plan from the arch key-stone. The arches remained self-supporting. This was the condition causing maximum deviation of the "line" arch from the arch axis, but the resulting calculated maximum stress was only about 50 per cent. greater than the calculated maximum stress in the arch-rings when the bridge was in service, and only a fraction of the safe compressive stress. In the process of demolition the mortar was found to be very good and the taper on the arch-stones regular. Following the longitudinal joints, and by drilling and splitting where joints could not be worked to, the outer strips of the arches were gradually racked back to the final position where the 12-foot wide middle strip was left.

In order to keep the work under control and to watch for any

indications of danger, permanent plumb-bobs were hung at both ends of all piers and abutments, and centre-lines pegged on the arches. The levels of piers and arch-crowns were also watched. Readings were taken periodically so that any movement of piers or arches would be noticed. No unexpected movements of any magnitude occurred.

The final loading of the two inner gantry-girders with the 12-foot-wide middle strip of the arch required some further precautions. The two inner gantry-girders in taking up their load would deflect about $1\frac{1}{4}$ inch, and the outer gantry-girders, through the action of their sway bracing, about half of this amount. It was necessary therefore to jack this deflection into the girders without causing any dynamic effect, and it was also expedient to avoid the unbalanced effect on a pier when the thrust of the remaining portion of an arch on one side only was removed.

The effect of the subsequent elastic recovery of the girders when the arch-voussoirs loading them were removed had also to be considered. This elastic recovery would severely strain and tend to break the end rods where they were tied down to the end voussoirs, since the latter, being in contact with those remaining attached to the piers, would be held down by friction and mortar-adhesion between the courses.

The calculated load at each suspender-rod when loaded with the middle strip of the arch averaged about 38 tons inclusive of steel centering. All sixteen suspender-rods to the inner gantry-girders were provided with jack-emplacements (*Figs. 13*, p. 494). Sixteen pairs of jacks were put into these emplacements, all connected to one pump and pressure-gauge.

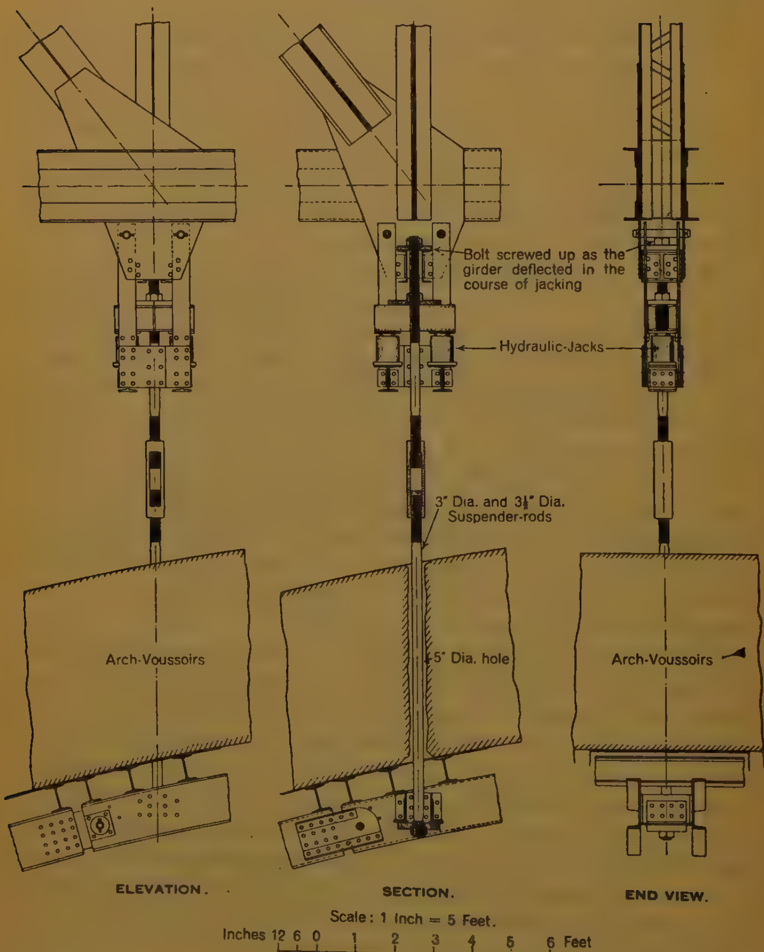
The procedure adopted in jacking was to commence at arch No. 8 and "half-jack" this arch, applying 21 tons to each rod. The second battery of jacks was in place on arch No. 9 and this arch was "fully-jacked" to 33 tons at each rod. The jacks from arch No. 9 were then placed at arch No. 7 and this arch half-jacked, followed by the full-jacking of arch No. 8, and so on until all the arches were fully jacked. The emplacements and jacks were then installed on the end rods only, so that the stress in these rods during the process of removal of arch-stones could be watched by periodically jacking them, and could be eased if they were found to be unduly stressed.

The removal of the middle strips then proceeded; a few stones were first removed from the centre, and then as those adjacent were removed they were stacked on the middle of the span so as to keep the gantry-girders deflected until the end courses were removed. The stacks of stone at the middle were then taken away.

As the stones were removed from the centre of the span the load in the end rods was found to increase, but not to any dangerous extent.

The masonry from the bridge has been removed to Harmondsworth in Middlesex.

Figs. 13.



SUSPENDED CENTERING AND JACKING GEAR.

Removal of Gantry-Girders.

The suspended centering was lowered on to barges in sections suited to the capacity of the two 6-ton gantry cranes. The outer

girders were removed by means of the gantry cranes working over the inner girders, and the inner girders themselves removed by cantilevering back. These girders had been designed with this method of removal in view, and had been provided with suitable top connecting links and heel thrust-blocks. On the north side spans Nos. 1 and 2 were balanced and removed from pier No. 1; on the south side span No. 9 will be removed from staging constructed beneath it.

Drilling Holes.

The drilling of the necessary holes in the masonry to the required degree of accuracy, and within the time during which the gantry-girders were being erected, presented some difficulty. In all, four hundred and thirty-six holes were drilled, with a total length of 4,234 feet. The diameters of the rods passing through the holes were $2\frac{1}{2}$ inches, 3 inches and $3\frac{1}{2}$ inches, and the nominal diameter of the holes 5 inches. The holes in the steel centering were 7 inches in diameter. The holes were centred by plumbing from the suspension-points on the girders after the bottom booms were erected. The difficulties experienced in keeping the holes true to line were mainly due to the obliquity of joints in the masonry passed through and to variations in the kind of stone met with. The drill tended to "run" in the direction of the lower resistance.

The drilling was done by The Demolition and Construction Company, Ltd., under a sub-contract. Seven Holman rotary percussion compressed-air drilling machines were used in the work. The air-supply at 80 lbs. per square inch was provided by three motor-driven stationary compressors each of 300 cubic feet per minute capacity and one portable petrol-driven compressor of 220 cubic feet per minute capacity. A 4-inch main laid under the footway of the temporary bridge, with branches at each pier, delivered air along the work from the compressor-house situated by the south abutment. Two receivers were installed, one near the compressor-house and one near the middle of the bridge. Each machine had an air-connection 1 inch in diameter and a water-connection $\frac{1}{2}$ inch in diameter. The drilling bits were of the cruciform type, measuring $5\frac{1}{4}$ inches across the blades, and were fixed to rods of $1\frac{1}{4}$ inch diameter. The rate of progress in drilling was very variable; its average was 8 feet per machine per day of two 10-hour shifts, the maximum being about 1 foot per hour.

Breaking-up of Concrete Aprons.

To safeguard the piers against the effects of scour, concrete aprons were provided in 1882 at all the piers except No. 1, which

had been incorporated in 1863 in the masonry construction of the Victoria Embankment. The aprons consisted of a 14-foot width of concrete slabs from 2 to 3 feet in thickness laid all round the pier and edged by 12-inch by 12-inch timber sheet-piles about 15 feet long. The top of the concrete aprons was generally at about river-bed level. The external measurements of the aprons were about 115 feet by 61 feet. In demolishing the pier-foundations it was decided to keep the cofferdams to the internal measurements of 95 feet by 38 feet. The aprons had therefore to be removed under water. Holes $1\frac{1}{2}$ inch in diameter spaced about $2\frac{1}{2}$ feet apart were drilled in them by means of $1\frac{1}{2}$-inch diameter drills fixed to 1-inch rods 20 feet in length operated from staging and passed through long guide-tubes. As each hole was drilled, 6-ounce gelignite cartridges were passed through these tubes into the hole. After blasting, the concrete was removed by grabs.

Cofferdams for Piers.

The steel sheet-piling used for the cofferdams consists of Dorman, Long's Krupp K.iii section, of length 50 feet and weight 32·56 lbs. per square foot. The walings are spaced at $17\frac{1}{2}$ feet between the first and second frames and 10 feet between the second and third frames. Both walings and frames are composed of steel joists fabricated mainly from material that had been used in the suspended centering for the arch demolition. Pier No. 5 has now [1] been completely demolished, and the work on pier No. 6 is proceeding.

Conclusion.

The bridge was not constructed as a national monument. It was originally built as a commercial undertaking for profit, and was given the name of " Waterloo " bridge as an afterthought.

It became a serious obstruction to river-traffic, and both its roadway and footpaths were too narrow for modern needs. Structurally it had failed. Appreciable settlement of its foundations is said to have occurred during construction. Protective measures against the effects of river-scour and wash from steamers became necessary, and in 1882 were provided by the construction of aprons round the piers. In 1923–24 the foundations of piers Nos. 5 and 6 were definitely failing and arches Nos. 5 and 6 distorted—both having to be propped—and the structure sagged $2\frac{1}{2}$ feet.

It could have been restored and, had it really been a national monument and not obstructing a commercial river, the heavy

[1] March, 1936.

expenditure would have been justified. Viewed from the Embankment it was a beautiful bridge, but passing under or over it was disappointing. It had had its day and lived its life. Apart from its failure and consequent propping, it had passed from utility to obstruction by the changing conditions around it. Its beauty remained, but was marred by a broken back. All Londoners, including the Authors, will regret the passing of Rennie's bridge, but a busy river is not a suitable site for an obsolete monumental structure.

The demolition of the old bridge to make way for replacement by one of more ample proportions has been accomplished so far without mishap. The precautions taken and the sure, if slow, methods followed have been successful in obviating the general collapse which must almost inevitably have occurred if any one of the arches had given way prematurely.

The remainder of the task, consisting in the removal of the piers from the river-bed, although rather behind programme-time, is proceeding according to plan, to clear the way for new construction. It may be of interest to state that it is intended to preserve a typical portion of the old bridge in the south abutment of the new bridge.

Acknowledgements.

The Authors desire to express their thanks to the London County Council for permission to present this Paper.

The Authors' firm were appointed engineers for the work in association with the London County Council's Chief Engineer, Mr. T. Peirson Frank, M. Inst. C.E., whose co-operation, together with that of the other officers of the Council, has been cordial and helpful throughout.

The principles of the scheme were drawn up under the direction of the Chief Engineer of the Council in the Bridges Division of his department, Mr. H. Firth, Assoc. M. Inst. C.E., being the Divisional Engineer.

In the office of the appointed engineers the work has been organized under Mr. S. M. S. Ram, B.A., Assoc. M. Inst. C.E., and the technical work has been done by Mr. R. P. Mears, B.A., M. Inst. C.E., and Mr. J. R. H. Otter, B.Sc., Assoc. M. Inst. C.E.

At the commencement of the work, Mr. D. L. Anderson, Assoc. M. Inst. C.E., of the London County Council's engineering staff, who had assisted in the preparation of the scheme, acted as Resident Engineer, but had subsequently to take up other duties. Mr. H. F. Nolans, M.A., B.A.I., Assoc. M. Inst. C.E., was appointed Resident Engineer and Mr. R. V. Allin, M. Inst. C.E., was appointed Assistant Resident Engineer.

Sir William Arrol and Company, Ltd., of Glasgow, are the

contractors for the work, and their agent at the site is Mr. J. S. Ramsay, who is assisted by Mr. H. P. Forge, Assoc. M. Inst. C.E.

The Paper is accompanied by eleven sheets of tracings and four photographs, from some of which Plate 1, the Figures in the text, and the half-tone page-plate have been prepared.

Discussion.

Sir GEORGE HUMPHREYS, Past-President, remarked that the thanks of The Institution were due to the Authors for placing on record a résumé of the salient features of the construction, failure and demolition of Waterloo bridge. Inasmuch as it was only during the last few weeks, however, and presumably since the compilation of the Paper, that the state of the piling under the foundations had been disclosed, the Authors would doubtless give some further information on that point. Sir George Humphreys.

Having been responsible for the bridge during the anxious period in 1923 and 1924, when the rate of settlement of piers Nos. 5 and 6 was disquieting, he had read the Paper with considerable interest, and especially the studied and temperate language in which the passing of the old bridge and the case for a structure more fitted to play its part under the stress of the traffic of future years were couched. He ventured to suggest that few would dissent from the ultimate decision which had been arrived at; and, looking back over events, he was sorry that he had wasted time in 1923 in attempting to put forward proposals to preserve the bridge. He was afraid that it had been rather a case of the heart running away with the head; but he felt at the time that the idea of a new bridge would arouse so much controversy that it would be well to avoid it. Events had proved that that surmise had been amply justified. A controversy raged on what was really a side-issue, namely, the feasibility or otherwise of underpinning, but no serious attempt was made to obtain the considered view of engineers as to whether it would be better for London to have a new bridge designed on modern lines or a reconditioning of the old one.

He had recently been privileged to inspect some of the old piling as it had been exposed. Most of the piles which he saw under pier No. 5 were round, and consisted of trees just as they had been felled. They had been driven with the butt uppermost, and in some cases the bark was left on. Very little adhesion remained between the bark and the tree in one which he had seen exposed. The exteriors of the piles had softened and could be easily pierced with a small penknife. That, he had been informed, was similar to the condition of the piling under Southwark bridge when it was demolished about the time of the Great War. He had been told by the contractor for that work that the timber in the piles was very good, except for the

Sir George Humphreys.

external portion, which was "all mush." He was of opinion that, by reason of that softening, the piles under Waterloo bridge could not have been capable for some time of acting as reliable supports, and the part which they played in sustaining the superincumbent load had been, he thought, small. The evidence afforded by the tilting of the masonry footings and the distortion of the timber platforms pointed to the load having been carried by the soil, with little aid from the piling; it had been suggested to him that it would be found that the piles under the corners of the platform had in fact come up, owing to the plastic action of the clay underneath.

The Authors were to be congratulated on having succeeded in taking down the arches without mishap. It was stated in the Paper that the use of suspended centering for that purpose had been considered by both Sir Basil Mott and himself; they had in fact advocated that method independently of each other. He had always had in mind the probability that the stability of the structure as a whole was dependent upon the arches being retained in their positions. Theoretically that was the case, but practice did not always agree exactly with theory, and unforeseen contingencies sometimes intervened. The knowledge that the Embankment arch was over the District Railway, running underneath the roadway with a not very strong roof, was in itself sufficient to dictate the taking of the most careful precautions. Opinions might differ as to whether all the Authors' precautions had been necessary, but evidently no risks had been run, and he could only say that in work of the kind in question it was far better to be safe than sorry.

It was significant that in the early stages, when piers Nos. 5 and 6 were settling, there were indications of movements in the parapets on each abutment. He had been interested to read the Authors' statement that it was doubtful whether the propping under arches Nos. 5 and 6 had ever carried any considerable load, and that pier No. 5 had ceased to settle before the propping was completed. It should be remembered, however, that the decision to prop those arches was made before it was known whether pier No. 5 would stop settling, and his chief anxiety had been to preserve the continuity of arches Nos. 5 and 6, so as to retain them in position as struts; that had been successfully achieved.

If he wished to be hypercritical, he would deplore the putting in hand of the demolition work before the plans were matured and a contract let for the new bridge. He felt that a considerable sum might have been saved if, for example, some of the stonework of the old bridge could have been crushed up and used for concrete in the new one. However, the views of the local authority had had to prevail. Directly it had been decided to have a new bridge, the

engineers had been pressed to make an immediate start, so that they were probably led to adopt a course of action which of their own initiative they would not have taken. Sir George Humphreys.

Mr. E. J. BUCKTON showed a number of lantern-slides illustrating the work described in the Paper, and exhibited samples of timber, granite, mortar, bricks, and other materials from the bridge, together with a small model of arches Nos. 5 and 6, arranged so that pier No. 5 could be lowered and the changing line of thrust through the masonry observed. Mr. Buckton.

Fig. 14 showed one of the latest drawings available of pier No. 5. The settlement of the pier was about 29 inches, and it had probably moved 4 to 8 inches to the south. It would be noticed that the outer piles were raking. There was no indication of that on any of the old diagrams which had been available. It would be seen that many of the piles were fractured and the footings on the left were completely cracked through. The pile on the extreme left was doing no work at all, as far as could be ascertained; the pier had moved away from it. There was a crack in the footing on the right, and from sections taken at other points it would be seen that in some cases the footings had separated away entirely.

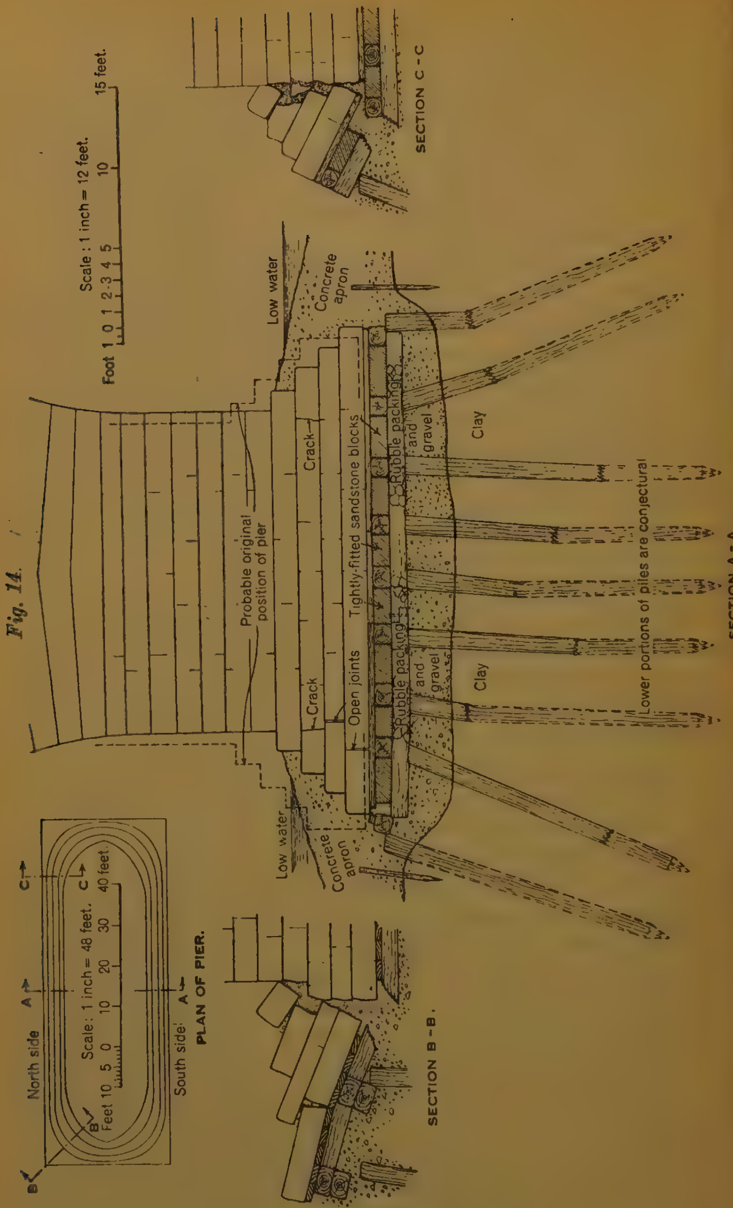
Fig. 15 was a similar diagram for pier No. 6. At that pier piles were all vertical. Most of them were drawn whole; they were not broken to the same extent as those under pier No. 5. The corner piles were probably above the original level, as mentioned by Sir George Humphreys.

Mr. T. PEIRSON FRANK observed that the remarks he proposed to make were not intended to convey the opinions of or to commit in any way the Council which he served. When he had come to London to take over his duties, Sir George Humphreys had shown him a few of the works for which he would be responsible, including the maintenance of the structure of Waterloo bridge, and he thought at the time that the bridge was about the worst or most perplexing inheritance that an engineer could possibly have. When he had been in London for a few months and had heard all the contentions about the retention or destruction of the bridge, he thought it would be wise to have nothing whatever to do with the controversy; he was, however, instructed to prepare a scheme for demolition. At different dates the London County Council had been consistent in saying that the bridge required to be replaced by a new one, and he thought it would be agreed, particularly in view of *Figs. 14* and *15* which Mr. Buckton had given (pp. 502 and 503) that the Council had been not only consistent but wise in its decision. Mr. Frank.

Several schemes for demolition were considered. *Fig. 16* (p. 504) represented one scheme which had been considered in some detail and

Mr. Buckton.

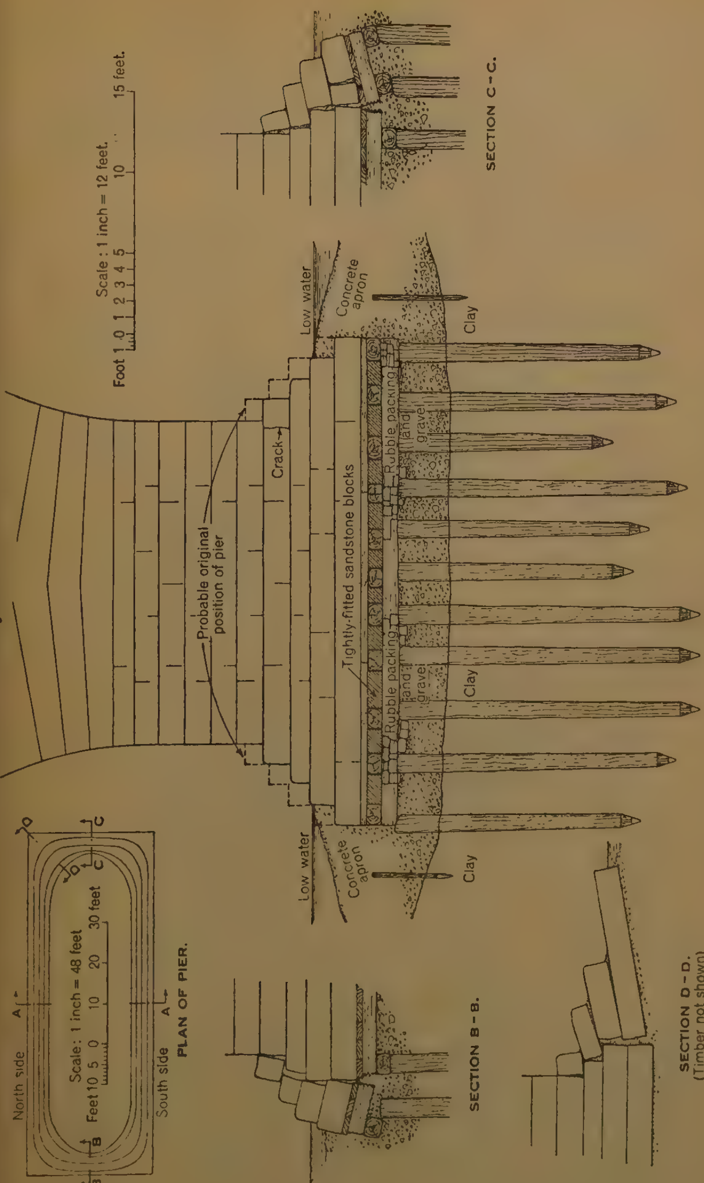
Fig. 14.



Pier No. 5: Conditions found during Demolition.

Mr. Buckton.

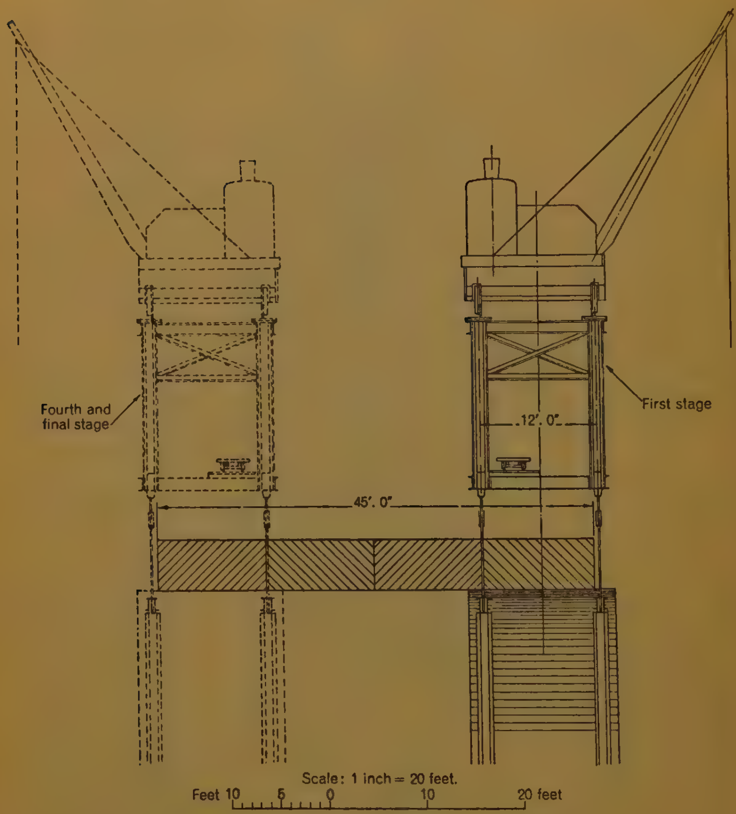
Fig. 15.



Pier No. 6: Conditions found during Demolition.

Mr. Frank. which was to employ a two-girder gantry and to traverse it across the width of the arch as each section was removed. That was considered carefully, but, in view of the total weights to be dealt with, he felt that the unbalanced thrusts which would arise involved too great a risk, and so a four-girder gantry was adopted. He then suggested that it might be better to make the suspender-rods as short as possible

Fig. 16.



by inclining them as shown in *Fig. 17*. From Fig. 12, Plate 1, it would be noticed that the outside suspender-rods passed through four of the voussoirs, which were large and difficult to deal with. It was stated in the Paper that some of the difficulties in drilling the holes true to line "were mainly due to the obliquity of joints in the masonry passed through and to variations in the kind of stone met with." The inclination of the suspender-rods would at least have removed part of that difficulty, because each rod would have

Mr Frank.

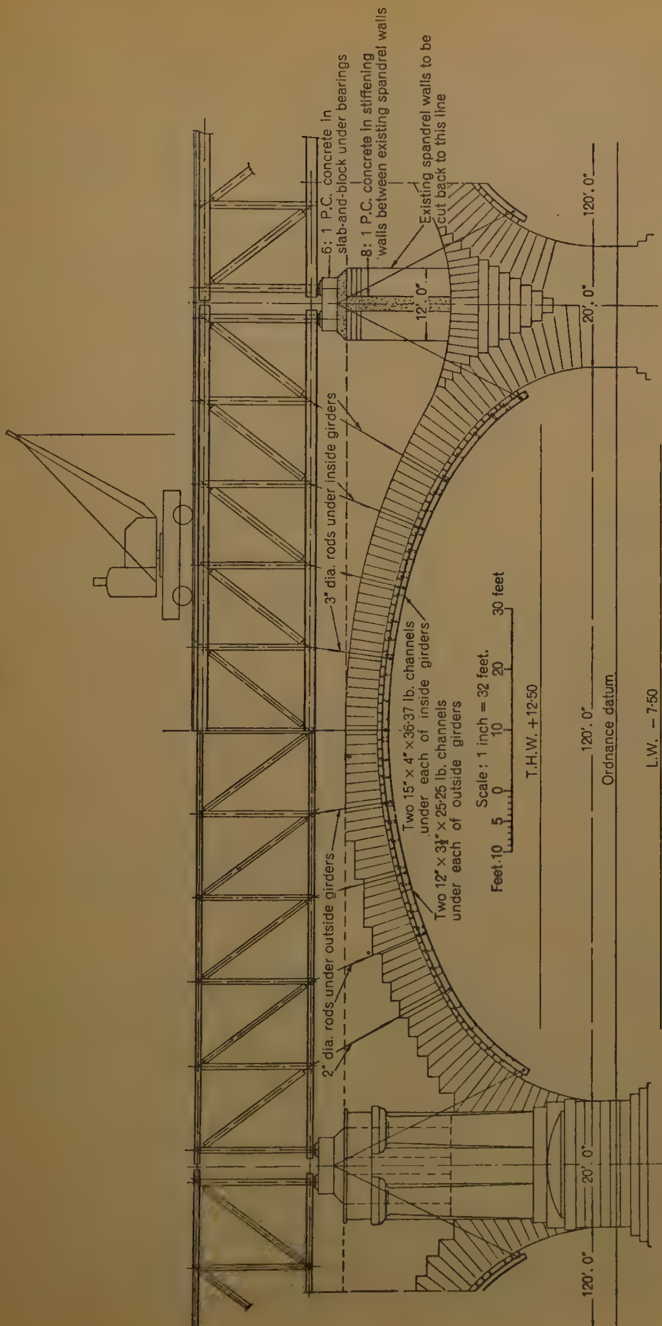


Fig. 17.

Mr. Frank. passed through only one voussoir, so that the drilling might have been easier. He agreed, however, that the transference of the load from the arches to the centering and on to the gantries would have been more difficult. He would emphasize that the details of the scheme had been prepared by the Authors ; he and his staff had only been associated with it. He mentioned the above suggestions only to show the various considerations involved, and not because he disagreed in any way with the methods adopted.

He might explain that the " value—cost " contract was of a very interesting type which had been largely used by Sir George Humphreys on other works, and particularly on large housing schemes, amounting in one case to over £13,000,000. The value of the work was approximately arrived at first. If there was to be open or any other type of competition, schedules were prepared giving certain values, and those invited to tender were asked to state whether they accepted that valuation or whether, and by what percentage, they wished it to be increased or decreased. They were also asked to state the percentage fee that they would require on the value of the work. From the tenders a comparative statement could quite readily be prepared showing their competitive positions. The main advantages of that type of contract were that the work could be put in hand quickly, that it was possible to vary the work during its progress (in the demolition of Waterloo bridge the work had of necessity to be varied as it proceeded), and that those for whom the work was done shared considerably in any savings that might be made. On the particular contract under discussion, he believed he was right in saying that the contractor was to receive a normal fee of 4 per cent. If, for example, the cost of the work were to exceed the value by more than 20 per cent., the contractor would receive only 2 per cent. Alternatively, if the cost were 20 per cent. below the value—the contractor having saved money—he would receive 6 per cent.

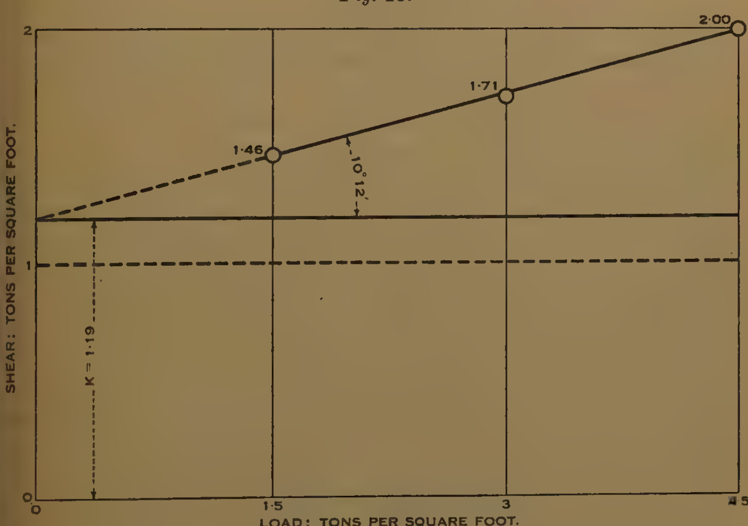
If he were asked why the bridge had failed, he would simply refer the inquirer to *Figs. 14* and *15*, which Mr. Buckton had given, to the statement at the foot of p. 476, and to Table I (p. 477). The latter gave the weights per superficial foot which had been brought to bear upon the ground below the piers, and paragraphs (*a*) and (*b*) below Table I gave reasons for the failure.

Sir George Humphreys had mentioned that there might have been some little hurry in getting on with the work, but that the order had to be carried out. Mr. Frank, however, was not quite sure of the correctness of the suggestion that a lower price might have been possible if more time had been taken, or if some of the old stone had been used for concrete. The stone used in Waterloo bridge wa

Cornish granite, which contained a larger percentage of mica than certain other granites, and he believed that more suitable aggregate and sand might be obtained than that produced by crushing material from the old bridge. Moreover, it was hoped to make use of a certain amount of the stone from the old bridge for facing purposes, probably for river walling. Mr. Frank.

Mr. H. G. Lloyd remarked that he had been engaged in testing materials in connection with the bridge. Samples of clay were taken directly they were exposed at the site of pier No. 5 at — 20·00 O.D., about 10 feet below the river-bed. *Fig. 18* showed the results of the Mr. Lloyd.

Fig. 18.



shearing tests on those samples when they were loaded at $1\frac{1}{2}$, 3 and $4\frac{1}{2}$ tons per square foot, and after those loads had been maintained for 24 hours. From those tests the value of K (the initial intensity of resistance to shear in Mr. A. L. Bell's formula) and α, showing the increase in resistance to shear with increased normal pressure measurable by the angle α, had been obtained. These were respectively $K = 1{\cdot}19$ and $\alpha = 10$ degrees 12 minutes. The average weight of the clay, w, was 127·5 lbs. per cubic foot, or 0·057 ton per cubic foot, and D, the depth of ground to the bed of the river, was 10 feet. The maximum permissible intensity r_2 of downward pressure on the foundations at the edge of the pier at depth D was given by Mr. Bell as a very long formula, but it could be transposed into the same form

Mr. Lloyd. as Rankine's formula, and from that he had been able to get a much simpler form, namely,

$$r_2 = wD\left(\frac{1+\sin\alpha}{1-\sin\alpha}\right)^2 + 2k\frac{2\cos\alpha}{(1-\sin\alpha)^2}.$$

Denoting $\left(\frac{1+\sin\alpha}{1-\sin\alpha}\right)^2$ by u, and $\frac{2\cos\alpha}{(1-\sin\alpha)^2}$ by v, the whole formula could be reduced to the simple expression $r_2 = wD.u + 2k.v$ within the limits of 0 and 22 degrees, and the values of u and v could be found from the equations

$$\log u = 0{\cdot}0311\alpha,$$
$$\log v = 0{\cdot}301 + 0{\cdot}017\alpha,$$

α being in degrees. The calculated value of r_2 for pier No. 5 was 8·08 tons per square foot, compared with a load of 8·45 tons per square foot as given in Table I, p. 477, when the cutwaters were excluded and only half the footings considered to be effective. That agreement was certainly remarkable.

The following test had been carried out to show the compression which took place on a sample of the clay in question. An enclosed cylinder of clay 3 inches in diameter and 4·55 inches deep was loaded at intervals of 6 hours (the time from high to low water) and 18 hours until at 3 days the load was 4·5 tons per square foot, when the compression amounted to 0·108 inch or 3·87 per cent. of the depth; at 7 days the compression was 0·127 inch or 4·16 per cent., and after 7 days no further compression took place.

The amount of the load added and removed at high water and low water at spring or neap tides could be calculated; from the records of Waterloo bridge tide-gauge for the three years 1930–32 the mean range of spring tides was 19·10 feet and that of neap tides was 15·70 feet. The difference between minimum and maximum load on the river bed at spring tides was thus 0·53 ton per square foot and at neap tides 0·44 ton per square foot; at exceptionally high tides it was 0·7 ton per square foot. The whole bed of the river was compressed as the tide rose, but the load on the clay just under the piers was correspondingly lightened by the effect of buoyancy; consequently the clay was subjected to a range of pressure at the edges of the piers equal to twice the change of hydrostatic pressure, or a maximum of 1·4 ton per square foot. It was therefore important to know what tidal conditions had been assumed in calculating the loads given in Table I, p. 477.

It would be realized that, as the material from pier No. 5 was removed, the pressure on the clay under it was decreased, and a change might be expected to occur in that clay. Perhaps the

compression which took place in the clay under the Thames as it was loaded could be recorded. Mr. Lloyd.

Mr. J. S. Wilson remarked that it was depressing to discuss the demolition of one of the finest masonry structures which had ever been built, and built by one of the most illustrious of English engineers. Mr. Wilson.

Few Papers read before The Institution had been devoted to works of demolition, and the present one was particularly welcome because of the interest and magnitude of the operations with which it dealt. He did not think that many arches of comparable size had been taken down with suspended centering, and the special precautions which had been taken for supporting the girders on the piers, on account of the state of the structure, were especially interesting.

Having been since the beginning in the thick of the controversy which had raged over the question of Waterloo bridge, he knew something of the psychology of the problem as well as of the purely engineering side. No one could have started on the demolition of Waterloo bridge without great trepidation. It had been stated repeatedly that the bridge was worn out, that the granite was shattered, that the timber was rotten, and, indeed, that the bridge was on the point of complete collapse. That view having been propagated for years, it could well be imagined that the work of demolition would be undertaken with the greatest caution and with the feeling that the dislodgement of one wrong stone would result in the Thames closing over the ruin. However, he himself had always believed that the masonry of the bridge was as sound as Rennie's description had indicated, and that the granite blocks had been chosen, shaped and bedded with great care. Even if the bridge had been built as a "commercial undertaking for profit," surely Rennie's record as an engineer called for a little more confidence; but the psychological effect of the various allegations had required 5-inch holes to be bored 20 feet down into the piers (p. 485) exploring for voids and bad joints, which had never been found. In spite of all the suspicion and the expectation of finding bad material and construction, the Paper proved and stated that the material, workmanship and construction were faultless; he felt that all present would agree with him that the fair name of a great engineer demanded an acknowledgement of that fact.

The criticism of Rennie's design by the Authors seemed to be much more severe than they could possibly have intended. Such criticism without making due allowance for the state of the art in Rennie's day, or the appliances which he had had at his command, was not helpful. An examination of the technical literature of 100 to 150 years ago would show that the theory and practice relating to

Mr. Wilson. arch construction was at that time in a chaotic condition. Bearing that point in mind he thought that everybody would agree that Waterloo bridge was a work of genius; in particular, the inverted arches over the piers were an outstanding feature.

The layer of clay filling over the bridge had been adversely criticized, but he thought that it had been good practice at the time of construction, and would be satisfactory at the present day. It had provided a waterproof layer, and had acted as a cushion for distributing the live load. Furthermore, it had been a cheap addition to the dead load on the arch, and had thus had the effect of reducing the deflection of the line of thrust caused by concentrated live loads.

He thought that the Authors had also been rather more severe with regard to the piles and the foundations than they could have intended to be. Table I, p. 477, showed that the total load to be carried on a pier was 10,500 tons, the total bearing area was 2,720 square feet, and there were 220 piles, but the way in which the pressure per square foot and the load per pile were given suggested that the pressure per square foot was 3·86 tons, and the load per pile 48 tons. Rennie could never have assumed that the load would all be carried by the piles, or that it would all be carried as a pressure under the masonry; he ought surely to be given credit for assuming that the piles would reduce the pressure per square foot under the masonry. If Rennie allowed for each taking an equal share, the load would be 1·93 ton per square foot on the clay and 24 tons on each pile; exception could hardly be taken to those figures. He was glad to see that the Authors allowed that the cutwaters took their share of the load during settlement, but he thought that they might have increased the area and number of piles considered to be effective under that condition and allowed for the corbelled extension. In that case, assuming that the load was divided between the piles and the pressure on the clay, the pressure would be less than 3 tons per square foot and the load per pile about 33 tons; that pressure and load, he maintained, were not excessive.

The failure of the foundations of Waterloo bridge could not be attributed to faulty design. Had Rennie known that the bed of the river would be dredged and scoured out to leave his footings well above river-bed level he would have provided for it. The historical notes given in the Paper prompted him to ask how it had come about that the bed of the river had been lowered so much without the bridge piers being adequately protected or underpinned. That was where, he thought, the competence of later engineers might be compared with Rennie's. It was on record that, while preparing to force grout under the masonry of pier No. 5—a remedial measure taken

in March, 1924, and not referred to in the Paper—when the boring tools pierced the timber platform they dropped 6 to 18 inches into a void. No one could suggest that Rennie would have built his foundations in that way. It appeared that the additional support provided by the ballast and the clay had disappeared, so that the bridge had been left supported on the piles, which were forced down. Similar piles driven into the bed of the river had carried a limiting load of about 50 tons each, and, with the bridge supported on the piles only, the load per pile of 48 tons stated in Table I appeared to agree quite well with that figure. It had been stated how sound the superstructure was, and it was evident that, if underpinned, the bridge could have been made sound throughout. In spite of the settlement of 28 inches, the arch was so strong, and its stability had such an enormous margin of safety, that it would have been a sound useful structure, even with the arch left as it was. He maintained that if the bridge had been underpinned, as it could have been, it would have been the strongest of the Thames bridges, not excepting the latest ones, as the masonry arch was the most perfect that could be desired, and the arch had a depth of 5 feet in the middle and 10 feet at the springing, with a comparatively small span. Mr. Wilson.

One other matter required to be mentioned in the discussion. On p. 478 it was stated that the Council of the Institution, when asked for their views on the proposal for underpinning the bridge " replied that in their view the London County Council would be well advised to act on the considered advice of their consultants. . . ." He suggested that the paragraph containing that statement should have included that part of the Council's reply which said that it was " outside their province to give a technical opinion on such a question." As it stood, the statement in the Paper suggested, as had been suggested far too often, that The Institution as an Institution endorsed the view that the bridge was worn out and should be pulled down to prevent it from falling down. If that correction were not made, outside bodies, when perplexed by some technical difficulty, would be encouraged to write to The Institution for the highest and most expert opinion in the land and to get it for the asking. Actually, The Institution had been quite correct in saying: " You have consulted eminent engineers, and, having consulted them, you should take their advice; it is not our business to advise you."

The Authors' criticism of the work of a man like Rennie was, he thought, a little unnecessary, when it was considered what had happened in the meantime. Taking, for instance, the piles which Mr. Buckton had shown in *Fig. 14* (p. 502), what had they had to withstand? When the pier had been built, the footings had been far below the river-bed, but later the bed had been so much lowered that the

Mr. Wilson.

piles had been deprived of lateral support. Then there was the possibility that the grouting process had caused the bursting-out sideways of the piles clear of the footing. It was perhaps significant that there was no sign of the piles having been pushed out beyond the platform under the pier shown in *Fig. 15* (p. 503), where no grouting had been done.

In conclusion, he desired to suggest that The Institution should take an interest in the preservation of the fine works of outstanding early members of the engineering profession. He had in mind Telford's great suspension bridge, about the reconstruction of which there were rumours, and he hoped that those interested in the work of The Institution's first President would exert their influence to keep extant that monument to his memory.

The Earl of Mayo.

The Right Hon. THE EARL OF MAYO said he was one of those who had always been in favour of the restoration of the old bridge, and remained unrepentant. In the introductory sentence of the present Paper the Authors alluded to the failure of some of the foundations. The actual cause of the failure of the bridge, however, was that the material which supported the foundations had been allowed to be scoured away, leaving the foundations without their proper support. The only wonder was that there had not been a far worse settlement due to those causes, and to the ill-judged recourse to grouting. To say that the bridge was obsolescent and incapable of adaptation to meet the growing demands of road and river traffic was incorrect, and his object in joining in the Discussion was to prevent certain statements being placed on record unchallenged.

Nothing could have been more unfortunate than the reply of the Council of The Institution in 1924 to the question as to whether it would be practicable to underpin the bridge. There had been a large body of opinion, both engineering and otherwise, in favour of the well-considered schemes of underpinning which had been prepared; but the reply had been tantamount to advising the London County Council against underpinning.

In the concluding paragraphs of the Paper the Authors spoke in disparaging terms of the old bridge, but to say that the bridge had become a serious obstruction to river traffic was an exaggeration, and to say that its roadway and footpaths were far too narrow was incorrect. The reason why the bridge had been demolished was not because it was not a national monument or because it was obstructing a commercial river; nor was it on account of the heavy expenditure (vastly less than that now being incurred), nor for any of the other reasons given by the Authors. The demolition had really been due to the determination of a small majority of the London County

Council who had long been intent upon its destruction, primarily with a view to bringing L.C.C. tramways across the new bridge. When that had been shown to be impossible, the tramway scheme had been shelved, but the fallacy had been imposed upon the public that the normal traffic necessitated a new and wider bridge; whereas the truth was that the vehicular traffic over the bridge had not appreciably increased during the past thirty or more years. The demolition of Waterloo bridge by the London County Council was the most wanton and shameful piece of vandalism ever perpetrated in London. The old bridge had been one of the most beautiful bridges ever built. The Earl of Mayo.

He had intended to make the same suggestion as Mr. Wilson, and now wished to support his proposition that the preservation of the works of great civil engineers should be recognized as a proper and legitimate object for The Institution's consideration. Surely the members could not but feel a sense of shame that they had done so little to honour and perpetuate the memory of the great Rennie.

Mr. H. P. Forge said that the demolition in 1913 of the old Southwark bridge had some aspects which were of interest in connection with Waterloo bridge. Southwark bridge was remarkably interesting from an engineering point of view and was a splendid example of bridge-construction in cast iron. It consisted of three segmental arch spans, the shore spans each being 210 feet clear, and the central span 240 feet clear, with two river piers, each 24 feet wide. Mr. Forge.

Appreciable settlement was said to have occurred during the construction of Waterloo bridge, probably caused by scour due to the obstruction of the eight piers, which occupied nearly 13 per cent. of the distance between abutments; it was presumably that which led Rennie to adopt fewer piers at Southwark, where the river was narrower; the obstruction there was only 6·8 per cent.

Southwark bridge was commenced in 1815, about $3\frac{1}{2}$ years after Waterloo bridge piers were started. The whole of the superstructure of Southwark bridge was made of cast iron, and the total load on one pier foundation was about 11,000 tons (about 500 tons more than in the case of Waterloo bridge), made up as follows: filling and roadway, 1,550 tons (14 per cent. of the total load); cast-iron ribs, spandrels, floor-plates, etc., 1,650 tons (15 per cent. of the total load); and masonry of pier, 7,800 tons (71 per cent. of the total load). The piers were solid masonry and were 36 feet wide by 89 feet long over the bottom course of the footings, giving an area of 2,750 square feet. The average load was therefore 4 tons per square foot, as compared with 3·86 tons per square foot for the foundations of Waterloo bridge (p. 477), or 31 tons per pile, assuming there were three hundred and fifty piles.

Mr. Forge.

The construction of the foundations (*Fig. 19*) was identical with that of the foundations for Waterloo bridge, but the timber used was of slightly heavier scantling, and the footings were thicker. There were no indications, as far as could be seen, of any appreciable bodily settlement, but there was evidence of the piers having tilted slightly outwards from the centre. The crown of the centre arch was flattened, and when the crown segments were dismantled the radial joints were found to be open about ¼ inch at the bottom. That was accounted for by the fact that the resultant of the arch thrusts was not in the centre of the pier, and produced a greater intensity of pressure on the land side of the pier than on the river side.

Apart from the defect he had mentioned, the bridge was in a remarkably good condition, after practically 100 years of life, and the Authors' remarks about the workmanship of Waterloo bridge applied with equal force to its contemporary, in the construction of which an entirely different material had been used. The arches of Southwark bridge were composed of eight ribs, each of thirteen segments in the form of flat cast-iron slabs which butted against the cross frames (also of cast iron), the latter being continuous right across the width of the arch. The ribs were held in place by long cast-iron, dovetail-shaped wedges, which were driven in and held them against the fillets on the cross frames. There were no bolts at all in the construction of the outer ribs; the inner ribs had a few bolts, but were held chiefly by the cast-iron keys. The floor-plates were also of cast iron. The only other load on the bridge was the earth filling.

Sir Clement Hindley.

Sir CLEMENT HINDLEY remarked that he had taken no part in the great controversy of the last few years about Waterloo bridge, though, since that magnificent bridge had been built by Rennie, one of the greatest exponents of engineering art, he had often wondered why it had been the architects who were up in arms about pulling it down. He had risen to speak because he had felt a certain amount of indignation when he had heard the London County Council accused of a shameful and wanton act of destruction. He wondered whether the speaker who had made such a remark expected to find any sympathy from the meeting, and he hoped that it would not come to be regarded as the considered opinion of The Institution. Had not engineers steadily to go ahead demolishing things that were out of date and that did not suit modern conditions? Was it not their business to restore natural means of communication, which The Institution's charter mentioned as one of their particular duties? How could London develop if the obstruction to land and river communication of Waterloo bridge were allowed to remain as designed in the year 1819?

He desired also to defend the Council of The Institution of 1924

Fig. 19.

FOUNDATIONS OF OLD SOUTHWARK BRIDGE DURING DEMOLITION.

Fig. 20.

FOUNDATIONS OF OLD VAUXHALL BRIDGE ABUTMENT DURING DEMOLITION.

(he had not been a member then) against the charge which had been made by a speaker who had said—or had assumed—that the Council had given advice to a local authority on a technical question. Exactly the reverse was the case. When the London County Council had said: "Please advise us on this question of underpinning," the Institution Council had replied: "We respectfully refer you to your own accredited consultant." What could possibly have been a more proper attitude on the part of the Council? Sir Clement Hindley.

One or two remarks had been made suggesting that the Authors had criticized Rennie. He felt that such a suggestion should be repudiated, as in fact, from beginning to end, the Paper was full of praise for Rennie's work. The attitude of a speaker who stated that the Authors had criticized Rennie's work, and then proceeded to demolish that criticism, was to be regretted; it went back to the old days of the fierce controversy which had now happily passed.

Mr. J. S. Wilson desired to clarify his statement with regard to the action of the London County Council. That body had applied to The Institution for advice on a technical matter, and the Institution had replied quite correctly, as he had stated and as Sir Clement Hindley had just emphasized. The London County Council, however, had misused that reply to give the impression that it was the definite opinion of The Institution that underpinning was impracticable and that the bridge could not be reinstated. Mr. Wilson.

Mr. William Muirhead remarked that much had been said about the question of the deterioration of the timber in the foundations, and he would like to mention some little-known matters regarding the foundation work of Rennie. The old Vauxhall bridge, though not mentioned in the Paper, was one of Rennie's great Thames bridges. When it had been taken down in 1898 the inclined foundations of the abutments were exposed (*Fig. 20*). The piles were somewhat similar to those under Waterloo bridge. When it had been required to settle the design of the foundations for the new abutments, Sir Maurice Fitzmaurice, then Engineer-in-Chief to the London County Council, had called in Sir Benjamin Baker. Sir Benjamin had inspected those piles and had said: "We will not take them out; they are better than anything we could put in, and we will put the new foundation on the top"—and the present Vauxhall bridge rested on Rennie's old piles in the abutments. Mr. Muirhead.

The piers of the old Vauxhall bridge had been of hard Midland stone, with cast-iron rivets, and could not have been removed without putting down a cofferdam. Timber cofferdams had therefore been built round the piers, and the masonry dismantled. The piers had been built on platforms with two piles of 12-inch by 12-inch criss-cross timbers. When each pier had been dismantled, with

Mr. Muirhead. several feet of masonry footings still resting on the timber, the valves in the cofferdam had been opened, and immediately Rennie's timber grid had floated to the surface, carrying its load of masonry. The end had then been taken out of the dam and the raft with the masonry on it towed down to Greenwich and there disconnected. He mentioned that in order to refute the statements that had been made about the timber being rotten under the water. He would also remark that Rennie's work in Waterloo bridge foundations should not be judged only by the condition of pier No. 5, which was now put forward as typical. More information would become available when the other piers were taken out.

It was remarked in the Paper (p. 478) that only by the demolition of the bridge had it become known how perfectly every arch-stone had been cut to taper and how solid and faultless had been the construction of the interior of the piers. That, to him, was a most astounding statement.

Mr. Gedye. Mr. N. G. Gedye said that it might be of interest to record that at the time of the controversy, and before evidence had been given before Lord Lee's Commission—(and he had had the honour to be one of the witnesses who had been called to give evidence on behalf of the Society for the Preservation of Ancient Buildings)—he had examined the foundations of No. 8 pier as far as had been possible by means of a small trial-pit, which had been opened out by the London County Council on the river side of No. 8 pier, and he had found that the timber which he had been able to see there was in good condition. It was slightly soft on the outside, and could be pricked with a pricker or knife, but it was quite hard inside.

Correspondence.

Mr. H. J. B. Harding observed that when the Royal Commission on Cross River Traffic in London was appointed in 1926, it issued a preliminary questionnaire to some of the principal witnesses. Question 3 read :— Mr. Harding.

"If the present bridge should have to be removed and not rebuilt,

(*a*) How long would the operation take ?
(*b*) How much would it cost ?
(*c*) To what extent would water-borne traffic be interfered with during the progress of the work ? "

In the evidence given before the Commission, one witness answered (*a*) 30 months, (*b*) £275,000, (*c*) none; another answered (*a*) 24 months after preliminary work, (*b*) £300,000, (*c*) no undue interference. On the other hand, two other witnesses gave (*a*) $3\frac{1}{2}$ to $4\frac{1}{2}$ years, (*b*) £500,000, (*c*) to some extent.

The first two witnesses were in favour of re-building, while the last two were in favour of retaining the old bridge. It now appeared as if the actual result would be midway between those estimates. Perhaps the Authors would now be able to answer those questions.

In the evidence given before the Royal Commission there was a report on the cementation work carried out under pier No. 5 in March, 1924, when the settlement first became severe, and before traffic was stopped, in order to attempt to arrest the settlement by injecting cement into the ballast supporting the pier. The report stated that nineteen holes $3\frac{3}{4}$ inches in diameter were drilled by diamond drilling through the piers, and then by crossbits through the timber beneath. "Below the timber gridwork, the drill tool invariably fell away by 6 to 18 inches, showing apparently that there is a cavity between the top of the ballast and the timber." The thickness of the Thames ballast under the pier, determined by means of the injection-pipes, was thought to be 4 feet to 5 feet. Only seven holes had been injected when the operations were stopped,

Mr. Harding. owing to a greatly increased subsidence in the pier. It would be of value if the Authors could give a short account of the conditions as actually revealed beneath that pier. The evidence stated that the actual mixture used consisted at first of 20 lbs. of cement to a tank of 75 gallons of water, and that it was increased by 20 lbs. of cement every four tanks until a consistency of 400 lbs. per tank was reached. The ultimate pressures reached were given as 150 lbs. per square inch and 250 lbs. per square inch, except in some cases where cement issued into the river. The quantity of cement injected was 23 tons 8 cwts., carried by 23,170 gallons of water, at a rate of 450 gallons per working hour. The initial consistency used was 1 volume of cement to 55 volumes of water and the final consistency 1 volume of cement to 2·72 volumes of water, the average consistency being 1 volume of cement to $6\frac{1}{2}$ volumes of water. The opinion was given in the evidence that the increased settlement was caused by the water used in injection lessening the adhesion of the piles in the ballast, before the cement had time to set, and it was suggested in that evidence that, in the light of events, it would have been better to grout up the voids under the pier before proceeding to treat the ballast. That had not been done, apparently on the grounds of speed, as re-drilling through the grout would have been necessary.

It had been suggested that the outer piles, some of which were found to be broken, had been fractured by the cement injection, bu that was not likely as the average pressure used appeared to hav been 150 lbs. per square inch, and the slurry would be free to pass al round the piles and probably to escape into the river in places. Fo grouting the cavities between the ballast and the pier, a consistenc of about $1\frac{1}{2}$ to $1\frac{3}{4}$ volumes of cement to 1 volume of water woul have been suitable. It would be interesting to know whethe the weak slurries, which were necessary to obtain travel of th cement in the ground, rose into the voids, and to what extent thos were filled. The amount of cement injected was very small, onl about $\frac{1}{2}$ cwt. per cubic yard of ballast under the pier, as the operatio was not completed.

Could the Authors describe what had been found under pier No. to what extent the ballast had been cemented, and what evidenc had been found of the travel of the cement and the effect on t voids under the pier?

Mr. Searle. Mr. A. B. Searle suggested that as the mortar used in Waterl bridge was described by the Authors as "exceedingly good" would be very useful if some further indication were publish showing how that conclusion had been reached. A chemical a mineralogical analysis of the mortar a short distance from the surfa

would be of great value to present and future bridge-builders, and he hoped that the Authors would be able to give such analyses. Mr. Searle.

The AUTHORS, in replying to the Discussion and Correspondence, observed that Sir George Humphreys' theory that the outer skin of the timber piles had deteriorated so that the piles had become more or less useless had been carefully considered and borne in mind during the preparation of the earlier-formulated scheme for underpinning. It was certainly a possibility, but in the Authors' opinion not a probability. Regarding the comparison which had been made with somewhat similar piles, where the sap-wood had been found to be mush, that was certainly not the case with Waterloo bridge; the outer wood of the old piles, though appreciably softer than the heart-wood, was certainly not mush. Particulars of a study of the condition of the timber had recently been published.[1] The Authors.

The suggestion that a saving might have been effected by placing a combined contract for the demolition of the old bridge and the reconstruction of the new bridge on the same site had little to support it. The demolition had been carried out by a reliable firm on a value—cost basis, and in work of that description there were so many unknown factors and so much risk that it was difficult to see a better or cheaper way of carrying it out. At the time when it was decided to demolish the old bridge the new bridge was an unknown quantity. During the demolition of the old bridge the designs for the new bridge had been developed, and it would, if desired, be possible to go to tender for the new bridge without the complications of combining works of very dissimilar characters. The possibility of crushing the old granite as an aggregate for the new bridge would certainly have offered no economy, and the material derived would have been less suitable than, and inferior to, the aggregates procurable in London.

Sir George had played a very responsible part in the most critical days of the old bridge. Although he had since retired from the office of Chief Engineer to the London County Council, his interest in the structure and the problem was as keen as ever.

Mr. Peirson Frank's suggestion that inclined suspender-rods might have been cheaper than vertical rods had been considered in the early days, but had not been adopted, as, although there would have been a saving on the drilling through the granite, there would have been more difficulty and expense in getting the suspended centering into position, in tightening up the suspender-rods, in jacking to pre-stress the temporary steelwork, and in checking

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[1] E. A. Rudge, "Studies in Old Timbers. IV. Waterloo Bridge Piles." Journal Soc. Chem. Ind., vol. lv. (1936), p. 221T.

The Authors.

and adjusting loads on the suspender-rods as the demolition proceeded.

Mr Frank had explained fairly fully the "value—cost" contract adopted for the demolition work. Under such a contract the contractor was paid the actual cost of the work plus a commission which varied according to the percentage by which the actual cost was above or below the "value" calculated on the measured quantity at rates agreed beforehand.

The remarks of Mr. J. S. Wilson and the Earl of Mayo came as a surprise. Both apparently considered that the Paper was an attack upon the fair name of Rennie. As evidence Mr. Wilson stated that the Authors' criticism was too severe, that the heavy clay filling under the roadway was good practice, and that the loads given in Table I were unfair, as he considered that Rennie had designed the structure for the loads on the piers to be carried partly by the piles and partly by the ground immediately under the rafts. Actually, the Paper merely stated facts, pointing out the severance of the cutwaters from the piers and the extent to which the footings were broken or weakened. To arrive at possible loads coming on the piles from the clay some assumptions had to be made. The Table given merely stated what the loads would be with certain assumptions.

The Earl of Mayo referred to disparaging terms used in the Paper, and said that the term "obsolescent" was incorrect. Actually the roadway was 27 feet 6 inches wide between the curbs, and each footpath 7 feet 6 inches wide. The river arches were of 120 feet clear span. The bridge stood on a sharp bend of the river where there was a strong set of the tide, and navigation for many years through the arches had been difficult; latterly it had been becoming much more so on account of the need for larger vessels to serve the industries, such as the Battersea and Fulham power-stations and Wandsworth gasworks, on the upstream side of the bridge. Footpaths 7 feet 6 inches wide on a river bridge in the heart of London were absurdly narrow, and the fact that the roadway was too narrow for even present-day traffic was proved by the intention, in the case of the old bridge being retained, of corbelling the footpaths so as to increase the width between curbs to 36 feet, although such corbelling would have caused the parapet to be out of the plane of the spandrel walls and would have thrown a deep shade line across the front of Rennie's structure. Whatever the effect of the corbelling would have been, it would certainly have appreciably changed the appearance of the bridge as designed and constructed by Rennie. The bridge was unquestionably obsolescent. The Paper stated on p. 49 that the bridge had passed from utility to obstruction by changing

conditions around it, and that should not be read as disparaging to Rennie. He built a bridge to meet the existing conditions and the future conditions as viewed in his time. The bridge had fulfilled its purpose, and the fact that it had taken 120 years to pass gradually from utility to obstruction was to the credit of the designer, as most bridges reached that stage in a much shorter period. The Authors.

Mr. Wilson's contention that, had the bridge been underpinned, it would have been the strongest bridge on the Thames on account of its great weight, and thus a most useful bridge, was certainly open to question. Great weight might mean strength or danger, and, as to usefulness, the underpinned bridge would have been far below the standard that ought nowadays to be adopted for a Thames bridge.

Mr. Muirhead's reference to the Authors' statement that the bridge was not constructed as a national monument, but was originally built as a commercial undertaking for profit, rather missed the point. All great engineering works were built for profit, either direct or indirect, but in discussions in Parliament Waterloo bridge had been referred to repeatedly as a national monument to those who fought at Waterloo, and it had been likened to the Cenotaph. The Cenotaph had not been built for profit; it was a specially designed, simple and beautiful structure erected on a carefully selected site in memory of those who fell in the Great War. The association of Waterloo bridge with the battle of Waterloo was no more than the association of a baby christened "Kimberley" in 1900 with the Boer War. Vauxhall bridge had not been one of Rennie's bridges. The design prepared by Rennie for a stone arch bridge had not been carried out.[1]

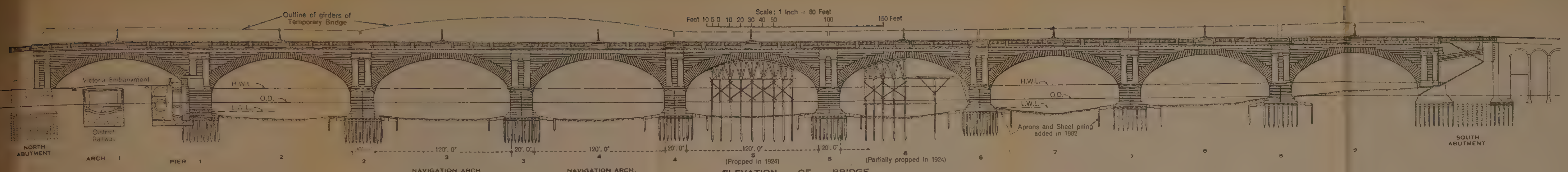
With regard to the points raised by Mr. Harding concerning the foundation of pier No. 5 and the effect of cementation on it, the following was found to be the arrangement of the foundation (*Fig. 14*, p. 502). A transverse timber was laid on top of each row of piles and the spaces between those transverse timbers filled with rubble. On top of the transverse timbers were longitudinal timbers, the spaces between them being filled with fitted blocks of stone. On top of them was a decking of timber about 4 inches thick on which the masonry of the pier was built. There was a layer of ballast, averaging about 3 feet in thickness, between the bottom of the pier foundation and the underlying London clay. Some small pieces of solidified cement were found in the vicinity of some of the boreholes which were drilled in 1924, but in no case was any effect noticeable more

[1] W. C. Copperthwaite, "Vauxhall Bridge, 1906." Minutes of Proceedings Inst. C.E., vol. clxix (1906), p. 268.

The Authors. than 18 inches away from the borehole, and there was no trace of any cementation in the gravel. During the demolition no signs were found of voids in the foundations of the pier, and nothing was then seen to account for the drill falling, as reported in 1924, from 6 to 18 inches after passing through the timber during the preparations for cementation.

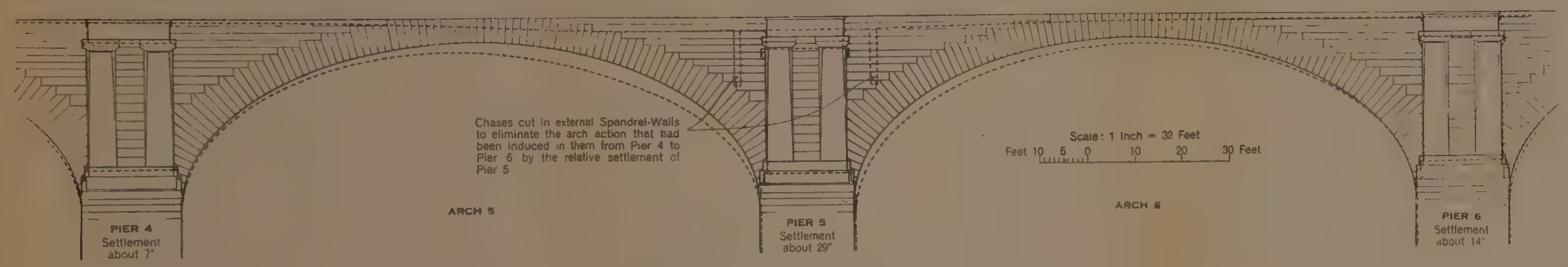
THE DEMOLITION OF WATERLOO BRIDGE.

FIG: 1.



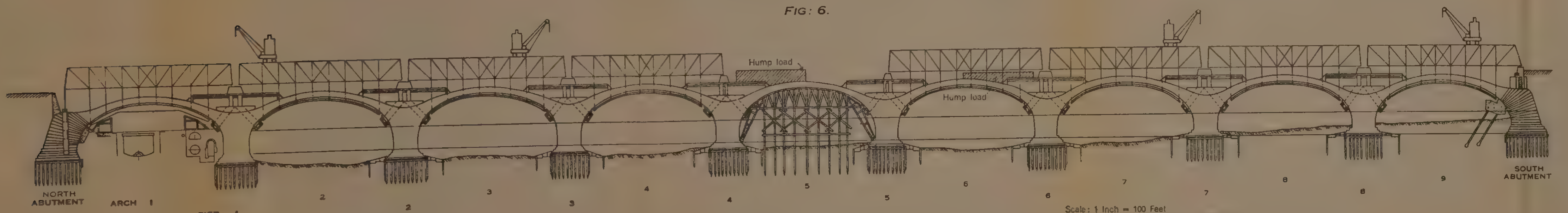
ELEVATION OF BRIDGE.

FIG: 4.



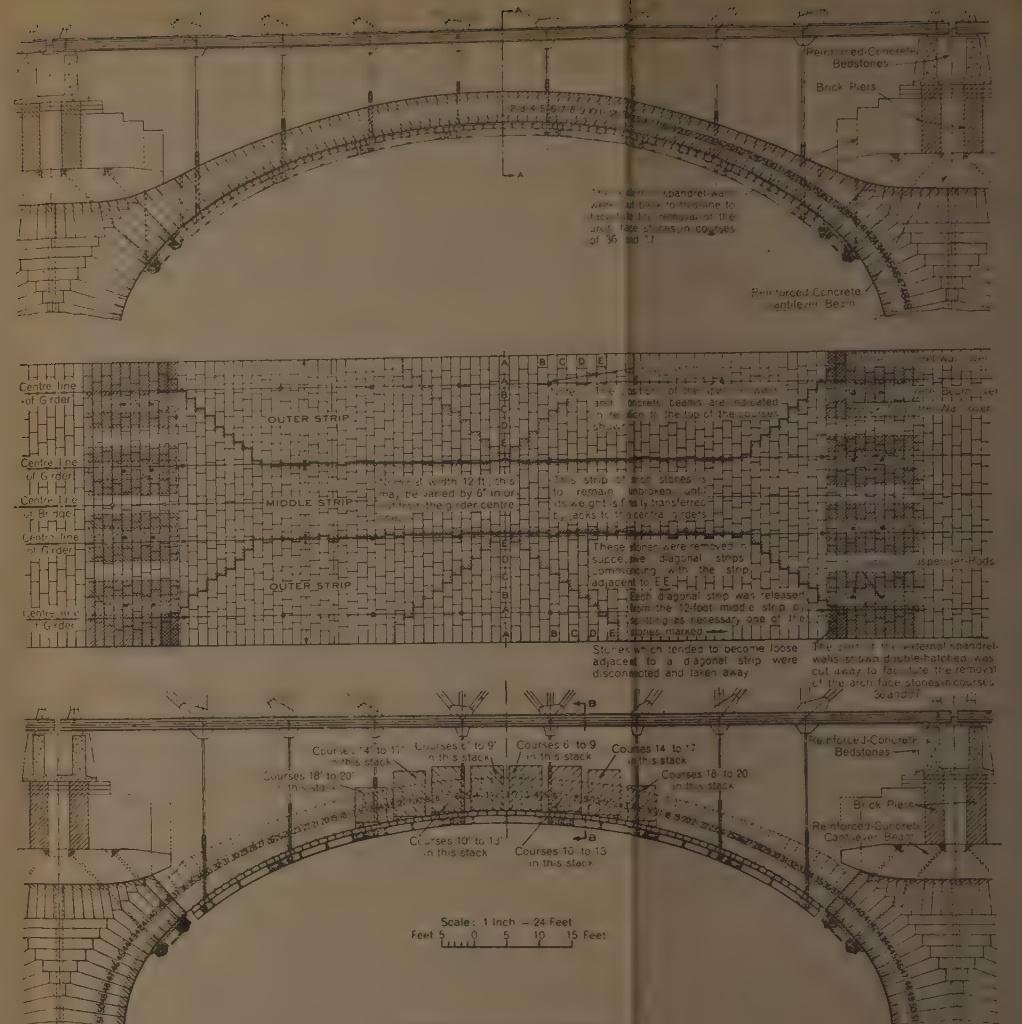
DISTORTION OF ARCHES NOS. 5 AND 6.

FIG: 6.



DEMOLITION OF ARCHES.

FIGS: 12



DEMOLITION OF ARCHES

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EXTRA MEETING.

5 May, 1936.

JOHN DUNCAN WATSON, President, in the Chair.

THE JAMES FORREST LECTURE, 1936.

The PRESIDENT stated that the James Forrest Lectures had been founded in 1891 in honour of James Forrest, who had been Secretary of The Institution from 1859 to 1896, and Honorary Secretary from 1896 until his death in 1917. The present Lecture was the forty-second of the series. A portrait in oils of James Forrest, together with a number of pieces of silver presented to him by Societies and individuals, were displayed in accordance with James Forrest's request, and might be seen outside the lecture theatre.

He had pleasure in introducing Mr. Relf, who was Superintendent of the Aerodynamics Department of the National Physical Laboratory, and who was about to deliver a Lecture which would undoubtedly be of the greatest value to some of the members. He (the President) knew very little about aeronautics, but he had had the pleasure of flying from the Mediterranean, where he had breakfasted, and having his tea in the Reform Club in the same afternoon, which showed how very quick and useful an aeroplane could be.

"Modern Developments in the Design of Aeroplanes."

By ERNEST FREDERICK RELF, F.R.S., A.R.C.Sc., F.R.Aë.S.

TABLE OF CONTENTS.

INTRODUCTION.

I CAN hardly say how much I appreciate the honour your Council has conferred on me in asking me to deliver the James Forrest Lecture this year. I realize fully the difficulty of the task that has been set me, for it is no easy matter to follow in the footsteps of the illustrious men who have previously addressed you on aeronautical subjects. I will, however, do the best I can to give you some idea of the development of aircraft since Professor R. V. Southwell delivered his excellent review in 1930,[1] and, in doing so, I will not fail to bear in mind the *leit motif* of this series of lectures, which is to trace, wherever possible, the interdependence of abstract science and engineering. My lecture is, in fact, arranged with that object mainly in view, for I shall endeavour to point out the advances that have been made in the technique of research methods and the nature of the new knowledge of aerodynamic phenomena which has resulted from them, and to show, as far as I can, how this new knowledge has reacted on the practical design of aircraft.

I should like to commence by calling your attention briefly to some of the outstanding advances of recent years, in order to preserve some continuity with the previous lectures of Dr. F. W. Lanchester [2] and Professor Southwell. We can then proceed to analyse in more detail the ways in which research and experience have led to those advances. Dr. Lanchester, in 1914, prophesied that advance in aircraft performance would come mainly from aerodynamic sources, and expressed his opinion that no great improvement could be expected from increased efficiency of the petrol engine. Professor Southwell, in his lecture, showed clearly that this was one of the rare occasions on which Dr. Lanchester's predictions had not been correct, and that, in the period under review, the greater part of the improvement in performance had been due to an almost phenomenal increase in the efficiency of the engine, achieved mainly by an enormous reduction in the weight per horse-power, but also by some improvement in the thermal efficiency. In the last 6 years, and

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[1] "Aeronautical Progress, 1914 to 1930." Minutes of Proceedings Inst. C.E., vol. 230 (1929–30, part 2), p. 333.

[2] "The Flying Machine from an Engineering Standpoint." *Ibid.*, vol. cxcviii (1913–14, part IV), p. 245.

especially very recently, the pendulum has swung over to Dr. Lanchester's side, and great advances have been made in the aerodynamic efficiency of aircraft, with comparatively little improvement in the power plant as such. This aerodynamic improvement is compounded of a number of more or less independent things ; better shapes for the main components, such as wings and bodies ; smoother surfaces, with the elimination of discontinuities that can upset air-flow ; abandoning of all external excrescences ; retraction of undercarriages ; closing of cockpits ; and better design of the means for cooling the engine. I shall come to speak of all these in some detail later on, but an idea of the magnitude of the changes that have occurred in recent years may be formed from Table I, in which the main characteristics of a good machine of 1930, the Fairey "Fox," are compared with those of two modern machines, the British "Comet," built by De Havilland, and the German Heinkel "He 70."

TABLE I.

	Fairey "Fox" 1930.	De Havilland "Comet" 1934.	Heinkel "He 70" 1935.
Weight : lbs.	4,750	5,550	7,390
Top speed : miles per hour at ground level.	152	235	235
Engine power : HP.	540	470	660
Engine weight : lbs. per HP.	1·8	2·1	1·8
Wing loading : lbs. per square foot	12·7	26·1	18·8
Power loading : lbs. per HP.	8·6	11·8	11·2
Drag coefficient (wing area)	0·0440	0·0197	0·0144
Drag coefficient ("wetted" area)	0·0145	0·0051	0·0046

From this Table it is evident that practically the whole improvement is due to better aerodynamic design, as exemplified by the figures in the last two lines. The quantity in the last line but one, defined as the profile drag coefficient, is obtained by subtracting the induced drag, namely the drag involved in the production of the lift, from the total drag, and reducing the difference to an absolute coefficient by dividing it by $\frac{1}{2}\rho V^2 A$, the product of the dynamic pressure and the wing area. It is thus a measure of the drag or head-resistance due to the friction of the air on the various surfaces plus that due to any eddying motion produced because the parts of the aeroplane are not perfectly streamlined. The last line of the Table contains a similar drag coefficient obtained by using the total "wetted" area of the whole machine instead of the wing area. If the streamlining were perfect, this figure might be expected to approximate to the known skin friction of a flat plate, and it accordingly serves as a rough indication of the extent of departure from a perfect form. We shall

need to refer again to these ideas later in the lecture. At the present moment, it is only necessary to note that either of the two forms of drag coefficient indicates a reduction in the case of the modern aeroplane to less than half the value for a good machine of 1930. This is a very great advance in a few years, and is really even more spectacular than it seems, because the greater part has occurred in the last two years. In the above comparison I have chosen modern machines of moderate horse-power. When the improved aerodynamic design of to-day is combined with the higher-powered modern engines, spectacular increases of speed are obtained. I am not able to give precise figures here, on account of the secrecy of performance data for the latest military aircraft. I can only say that Professor Southwell's figure of about 170 miles per hour for the fighter of 1930 would be replaced to-day by a figure above 300 miles per hour. This can be deduced from the Table, for if the Heinkel machine were given the power loading of a modern military machine, say 6 lbs. per horse-power, its speed would be $235 \times \sqrt[3]{\frac{11 \cdot 2}{6}}$, which is nearly 300 miles per hour. We shall deal first with flight at high speed, since it is this characteristic which gives the aeroplane its chief advantage over other means of locomotion, whether it be for the transit of passengers and goods in peace, or for rapid attack in war. Afterwards I shall proceed to consider the reactions of increased speed on other characteristics of the aircraft, such as the necessity of being able to fly slowly in order to land, and of providing adequate stability and control in any manœuvres the aircraft must perform.

Aerodynamically speaking, the aeroplane consists mainly of two parts—the wings and the fuselage. The wings provide the lift to balance the machine's weight and so make horizontal flight possible; the fuselage houses the passengers or goods which are to be carried. The production of lift by the wings was considered by Professor Southwell and by Dr. Lanchester, and the former pointed out that it was not necessarily associated with drag. In fact, if the span of the wings could be made infinitely great, the production of lift would involve no drag. With a finite span, however, there is a drag associated with lift, usually called the induced drag, and this becomes greater as the ratio of span to chord, known as the aspect ratio, becomes smaller. In addition to the induced drag, there is a drag due to the fact that air is a viscous fluid and so exerts a frictional force on a body moving through it. In the case of high-speed flight this latter part of the drag, commonly known as profile drag, is the only important factor, even if the aspect ratio is small. This is so because the induced drag, for a given aspect ratio, varies inversely

as the square of the speed of flight. How shall we reconcile all this with Dr. Lanchester's statement that maximum efficiency of the wing involves great aspect ratio? The answer is that the modern aeroplane at high speeds cannot fly under the conditions of maximum wing efficiency, namely maximum lift/drag ratio. If the wing area were made such that top speed occurred at the angle of incidence appropriate to maximum lift/drag, then the minimum possible flight speed would be so high that landing would be impossible. We are accordingly forced to use a larger wing area for the sake of landing, and this causes high-speed flight to occur at a much smaller angle of incidence than that of maximum lift/drag. The induced drag then becomes small—it is about 5 per cent. of the total drag on a modern fighter—and we need only consider the drag due to frictional actions at the surfaces in contact with the air.

The Theory of Dynamical Similarity and the Reynolds Number.

Before I can proceed to discuss what we have learned in recent years about the frictional forces on bodies moving in a fluid, I must refer briefly to the theory of dynamical similarity and introduce the important factor now commonly known as the Reynolds number. As a great deal of our data are obtained on models, we may conveniently study, at the same time, the conditions under which model-results may be applied to full-scale prediction. One of the oldest applications of model-testing, and one that affords a good illustration of the laws of comparison, is the use of ship-models towed in a long tank to measure the force necessary to move them at different speeds. It can be shown that the wave-making resistance for geometrically similar forms will be exactly proportional to the square of the speed and also to the square of the length provided that the speed is proportional to the square root of the length (Froude's Law). Thus a ship-model made to a scale of $\frac{1}{64}$ must be tested at $\frac{1}{8}$ of the speed of the ship, and by covering a suitable range of model speeds, the wave-making resistance of the ship can be predicted over the range of speeds at which it will operate. The law in this case is a convenient one, for it involves a speed for the model lower than that of the ship itself. The above law may be deduced very simply from the assumption that the wave-making resistance depends only on :—

(1) The scale of the model (length l).
(2) The speed (v).
(3) The density of the fluid (ρ).
(4) The value of gravity (g).

It is only necessary to apply the condition that the dimensions of both sides of a physical equation in mass, length, and time must be the same, in order to deduce the general law for the wave-making resistance (F) :—

$$F = \rho v^2 l^2 . f(v^2/lg)$$

where f is an undetermined function, the form of which cannot be found by dimensional analysis. For the purpose in view it is not necessary to know the form of this function, as it is evident that if v^2/lg is made the same for both model and full-scale ships, then the function will have the same value in both cases and F will be exactly proportional to $\rho v^2 l^2$. Since g is the same for model and full scale, the law of comparison simplifies to $v^2/l =$ a constant or $v \propto \sqrt{l}$ as stated above.

This illustration is given on account of its familiarity and of the simple and convenient form of the resulting law of comparison. When we come to consider the resistance of a body completely immersed in a fluid and far below the free surface, so that wave-making does not occur, the law does not take so convenient a form. The value of gravity is now no longer involved, and the dominating physical properties of the fluid which determine the motion are its density (ρ) and viscosity (μ). Application of the dimensional theory now leads to the expression

$$F = \rho v^2 l^2 . f(vl\rho/\mu)$$

and if we experiment on a model of an aeroplane in an ordinary wind tunnel, so that ρ and μ are substantially the same for model and full scale, we find that we must keep vl constant to obtain dynamic similarity of flow in the two cases. This cannot be done in practice since it involves very high speeds for the test of the models. For example, a $\frac{1}{10}$ scale model of a machine flying at only 70 miles per hour would have to be tested at about 1,000 feet per second. Even if this enormous speed could be produced in a wind tunnel, the above theory would cease to be valid, because the compressibility of the air, neglected in obtaining the law of comparison, would become important at speeds approaching the speed of sound in air.

There is, however, one way in which the law of comparison can be satisfied in the laboratory without involving the complication of compressibility effects. It happens that the viscosity (μ) of a gas is sensibly independent of the pressure of the gas. The density (ρ) varies directly as the pressure. It follows that if we can test a model in a current of air compressed to say 20 atmospheres pressure, the twenty-fold increase of ρ will compensate for a twenty-fold decrease of the product vl. We can then test our $\frac{1}{10}$ scale model at

half the flight-speed, or about 50 feet per second in the case cited, and obtain exact dynamic similarity of flow. This principle is used in the compressed-air tunnel at the National Physical Laboratory, results from which I shall use later in developing my subject.

In the absence of special facilities, such as the above tunnel, we can only explore the variation of the force coefficient $F/\rho v^2 l^2$ over as large a range as possible of the parameter $vl\rho/\mu$, and extrapolate as best we may to the full-scale value of the latter quantity. Comparison with actual flight tests will give us an idea of the accuracy of the extrapolation. The parameter $vl\rho/\mu$ is known as the Reynolds number, in honour of Osborne Reynolds and his pioneer work on fluid flow, and is generally denoted by the symbol R. It has, as will be gathered from the above, a fundamental importance in relation to the aerodynamical phenomena involved in flight.

The Frictional Resistance to Motion in a Fluid.

We are now in a position to discuss the frictional resistance to motion in a fluid, and we will start by considering the simplest case; that of a flat plate of infinitesimal thickness moving in its own plane. If we were able to solve the complete equations of motion of a viscous fluid, we should be able to deduce the resistance of a body of any shape at any Reynolds number. Hitherto these equations have only been solved, even for a simple case like the flat plate, for low values of Reynolds number at which certain terms in the equations can be neglected. This process applied to the flat plate leads to the well-known expression for the resistance, due to Blasius [1] :—

$$F = 1{\cdot}327 \rho V^2 l R^{-\frac{1}{2}}$$

In this solution the flow near the plate is described as "laminar," that is, every particle of the fluid moves in a straight line very nearly parallel to the plate. The retarded layer of fluid close to the plate is termed the "boundary layer," and is very thin near the front edge of the plate but gradually increases in thickness as we proceed downstream along the length of the plate. The theory tells us the nature of the velocity distribution across the thickness of the boundary layer at any distance from the front edge, and both this and the resistance have been experimentally measured and found to agree with the theoretical deductions.

I have said that the above theory only applies at low Reynolds numbers, that is, only at low speeds or at short distances from the front edge of a plate. What happens when the Reynolds number is increased? To obtain an answer we must resort to experiment,

[1] *Zeits. für Mathematik und Physik*, vol. 56 (1908), p. 1.

since the solution of the complete equations of flow, even for this simple case, has so far proved beyond the powers of the mathematician. We are indebted to Professor J. M. Burgers, of Delft, for the first extensive experimental study of the flow near a flat plate. He measured the velocity near the plate by means of a hot-wire anemometer, which consists essentially of a very fine wire, electrically heated, and placed with its length at right angles to the direction of the air flow. The loss of heat from such a wire depends on the speed of flow, and the electrical power needed to maintain it at a given temperature (conveniently defined by a given value of its resistance) is therefore a measure of the air speed. Professor Burgers found that at a certain distance from the front edge of his plate (which he mounted in a wind tunnel) a change occurred in the flow close to the plate. Instead of being quite steady, the flow became disturbed or turbulent, and the nature of the velocity distribution in the boundary layer changed also, the gradient of velocity very close to the surface becoming much greater, with a consequent increase in the intensity of the surface friction. He found, moreover, that this change to turbulent flow occurred at different points along the plate as the air-speed in the tunnel was changed, and that the product of speed and distance along the plate was roughly constant. In other words, the change occurred always at approximately the same Reynolds number, the mean value of this Reynolds number found by Professor Burgers being 3×10^5. *Fig. 1* shows turbulent flow close to a flat plate, photographed by means of smoke from drops of titanium tetrachloride placed on the surface.

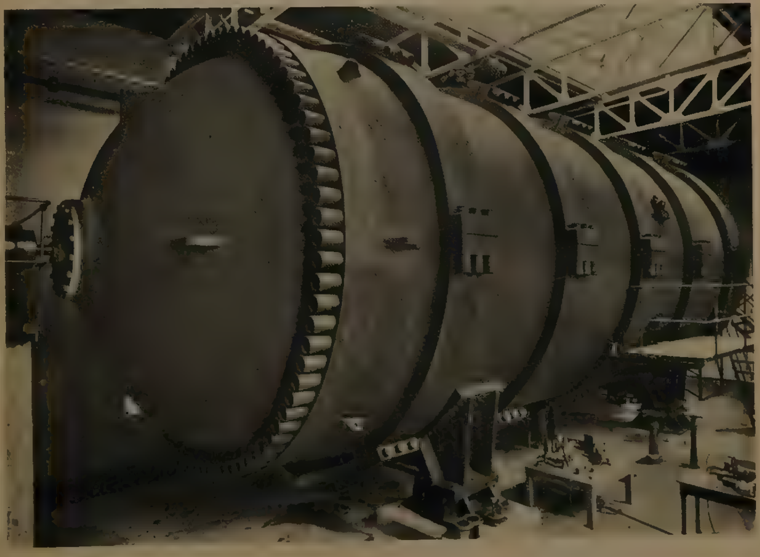
We can now form a physical picture of what happens near the surface of a long body of good form moving in a viscous fluid. At very low speeds the flow in the boundary layer is laminar, the layer increasing in thickness as we go along the body, and the gradient of velocity at the surface decreasing as the layer thickens. At a certain speed the boundary layer becomes turbulent at a point near the downstream end of the body, and as the speed is further increased, this "transition" point approaches the forward end of the body until at a very high speed it is close to the forward end. As the "turbulent" skin friction is higher than the "laminar," the first effect of the onset of turbulence will be to increase the overall resistance, but when the transition point gets near the forward end of the body, and ceases to move forward so quickly with increasing speed, this increase of overall resistance will be checked.

In *Fig. 2* (p. 531) I have collected typical curves showing the resistance of an aeroplane wing and a streamline body over a wide range of Reynolds numbers. The two curves marked A and B give the resistance coefficient for a flat plate when the boundary layer is

Fig. 1.

TURBULENT BOUNDARY-LAYER FLOW ON A FLAT PLATE.

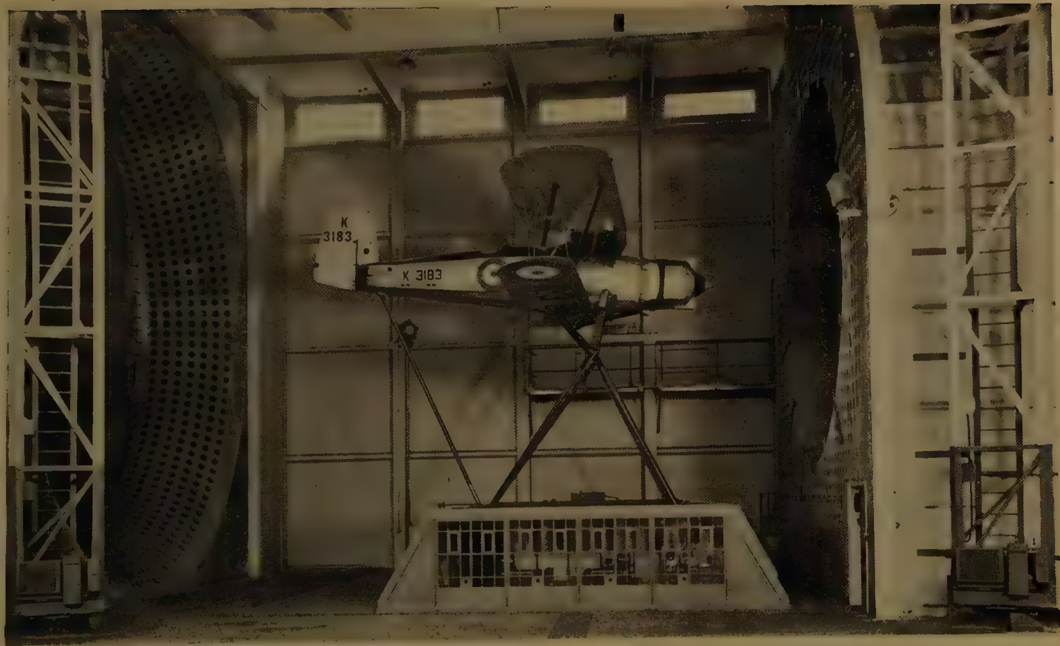
Fig. 3.



COMPRESSED-AIR TUNNEL.

J. F. Lect.

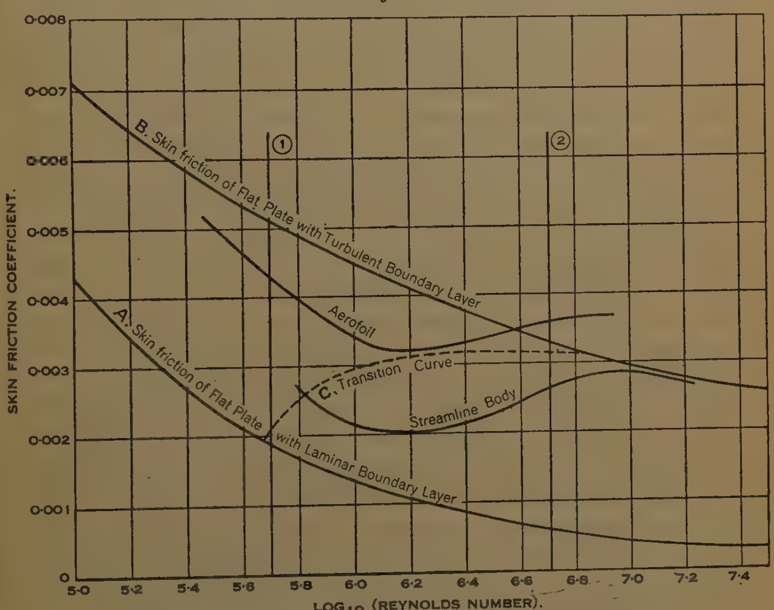
Fig. 12.



24-FOOT WIND-TUNNEL AT ROYAL AIRCRAFT ESTABLISHMENT.

respectively laminar and turbulent all along the plate. The curve A may be calculated from Blasius' theory, mentioned earlier; the curve B is experimental. At low Reynolds numbers one would expect the resistance, or "drag," as it is usually called, of bodies of good streamline shape to lie near the curve A; at very high Reynolds numbers, when the whole of the boundary layer has become turbulent, they would be expected to lie near the curve B; and at intermediate values, where the boundary layer is laminar near the front of the body but turbulent farther back, the drag would be expected to

Fig. 2.



Drag Variation with Reynolds Number on Bodies of Different Forms.

lie between the two curves. This does in fact occur in the case of flat plates where the whole of the drag is due to surface friction, and experiments give a curve which follows the laminar curve A up to a certain point, and then follows a "transition" curve such as C in the figure, reaching the turbulent curve B at very high Reynolds numbers. When, however, we come to a wing section or an airship form having finite thickness and a curved contour, the behaviour is complicated by the fact that the velocity just outside the boundary layer is not now constant as it is for the flat plate, and by the introduction of a drag due to the resultant along the wind direction

of the normal pressures on the surface. At very high Reynolds numbers the boundary layer is very thin, even at the rearward end of the body, and the flow outside the boundary layer approximates very closely to that given by potential theory for a perfect fluid. One would accordingly expect the drag due to normal pressures, or form drag, as it is usually called, to be small, approximating in fact to the zero drag given by the perfect fluid solution. The curves in *Fig. 2* show that this is actually the case, the drag of both aerofoil and airship forms tending to fall near to the turbulent skin friction curve B at the highest Reynolds numbers. At low Reynolds numbers the boundary layer becomes thicker, and may even break away from the surface of the body altogether, as was explained at some length by Professor Southwell in his 1930 lecture. Under these conditions one would expect a higher "form" drag. But at these lower Reynolds numbers the boundary layer will in general be partly laminar and partly turbulent, so that the drag due to skin friction will tend to lie between the curves A and B. The addition of a form drag which is a function both of the Reynolds number and of the form of the body gives rise to curves of a variety of shapes lying in general in the region between curves A and B. Only when the body becomes very thick in comparison with its length does the form drag become so large that the drag curve lies well above the curve B.

Now in the transition region, say between the ordinates (1) and (2) shown in *Fig. 2*, the drag is largely affected by the position along the body at which the boundary layer flow changes from laminar to turbulent, in other words, the position of the "transition" point. We should expect this point to be affected by the amount of turbulence present in the air stream of a wind tunnel. This is found to be the case, the drag rising as the turbulence of the stream is increased. For this reason tests of the same models in different types of wind tunnel may yield quite different drags in this "transition" region of Reynolds number. If, however, we make our tests at Reynolds numbers higher than that of the ordinate (2) in *Fig. 2* we should expect much less variation of drag with turbulence in the stream, because now the boundary layer has become almost wholly turbulent, and an extra disturbance from the stream turbulence would not be expected to affect the boundary layer flow very much.

The ideas outlined above have given us a much clearer conception of the nature of the variations of skin-friction drag with Reynolds number, and of the effects likely to be produced by changes of turbulence in the stream. They suggest a vitally important practical conclusion that at the very high Reynolds numbers involved in flight

the drag, whether of wings or fuselage, ought, with a good shape, to approximate to the value given by the turbulent skin-friction curve. We shall see later how nearly this value has been approached in modern practice.

Compressed-Air Wind Tunnels.

In the early days of flying, experiments in quite primitive wind tunnels were sufficient to give results that enabled designers to choose reasonably good shapes for the parts of their aeroplanes. At the present time, design calculations are carried out with much greater refinement, and the need is continually being felt for more and more accurate data. In particular, the changes of aerodynamic forces with Reynolds number, generally described by the term "scale effect," which might have been classed as small 10 years or so ago, are now important. It is here that the compressed-air tunnel is invaluable in leading us to improved aircraft design. There are two tunnels of this kind now in existence, the variable-density tunnel built in America in 1922, and that built at the National Physical Laboratory in 1930. I shall describe the latter tunnel (*Fig. 3*, facing p. 530) very briefly. It consists of a 6-foot circular wind tunnel of the return flow type, put inside a cylindrical steel shell with hemispherical ends. The shell is about 2½ inches thick and weighs nearly 300 tons. It is strong enough to stand an internal working pressure of 25 atmospheres (over 350 lbs. per square inch), and at this pressure the 500 HP. provided to drive the airscrew circulating the air will give an air speed of about 90 feet per second. This corresponds to a Reynolds number of about 9 million on a model wing of 8-inch chord, or, put in another way, to a flight speed of 165 miles per hour on an aeroplane having wings of 6-foot chord. The great merit of this wind tunnel is that it enables aerodynamic phenomena to be studied, not only under full-scale conditions, but also over the whole range of Reynolds numbers between that in an ordinary atmospheric tunnel and that of full-scale flight. It thus offers a much better opportunity of understanding the changes which occur than would be the case if only the two ends of the Reynolds number range were available, as in earlier days. There is not time now to attempt even a brief description of the details and method of use of this fascinating piece of research equipment ; those who are interested will find descriptions elsewhere,[1] including an account of the way in which the aerodynamic

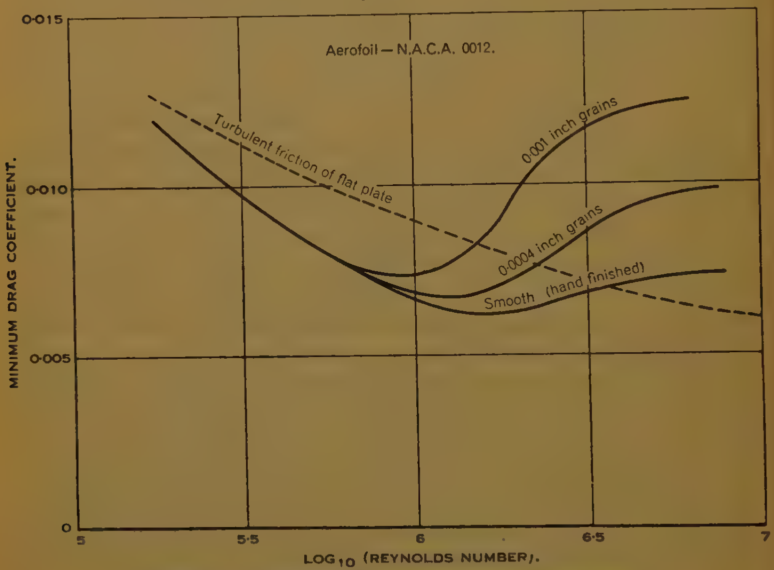
[1] E. F. Relf, "The Compressed-Air Wind Tunnel of the National Physical Laboratory," *Engineering*, vol. 132 (1931), p. 428.
„ „ "Results from the Compressed Air Tunnel," Journal Roy. Aē. Soc., vol. xxxix (1935), p. 1.

forces on models are measured outside the tunnel by means of electric distant-reading balances.

The Effect of Surface Roughness.

Let us glance at some of the results that have been obtained by the use of the compressed-air tunnel. The curve of aerofoil drag given in *Fig. 2* was from this source, and shows how the whole range of Reynolds number can be covered by using various air-pressures in the tunnel. This drag curve applies to a wing with a smooth surface, the model having been made of metal with almost a polished

Fig. 4.



Effect of Surface Roughness on Wing-Drag.

finish. What would happen if the surface were slightly roughened, to represent, if you like, the roughness of a fabric-covered full-scale wing? This is a vital question to the designer, for the provision of a really smooth surface is no easy matter, even with metal-covered wings. We had some guidance as to what to expect from some German experiments on the resistance to flow in rough pipes, which indicated that for a given degree of roughness there was a certain Reynolds number above which the resistance progressively increased, but below which there was no effect. Tests on aerofoils, roughened by painting them with a mixture of lacquer and carborundum dust, led to the results depicted in *Fig. 4.* Two grades of dust were used,

the average size of the particles being 0·0004 inch and 0·001 inch respectively. It will be seen from the curves that neither form of roughening has any effect on the drag at low Reynolds numbers, so that a test of this kind made in an ordinary wind tunnel would give a negative result. At the full-scale end of the range, however, the drag is increased by about 35 per cent., and 70 per cent. for the two degrees of roughness, an amount large enough to be extremely important even for the finer roughening. From these and similar results I have derived an approximate formula which will tell the designer whether in any specific case a distributed roughness of this type is sufficient to affect drag. The formula is :—

$$h = 100/R$$

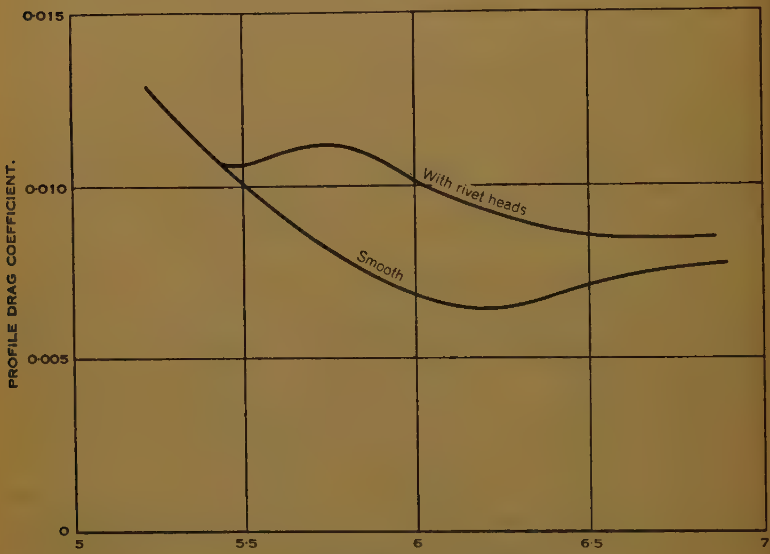
where h is the height of the excrescences in millionths of the wing chord, and R is the Reynolds number in millions. For a small aeroplane near top speed, R may be 10 million and then $h = 10$ millionths of the chord or about 1/1000 inch on an 8-foot chord. This shows strikingly how small is the degree of roughness that begins to affect the drag of large surfaces moving at high speeds.

We can explain why this effect occurs. The idea of the turbulent boundary layer has already been made clear to you. Very close indeed to the surface the turbulence is much reduced by the predominating effect of viscosity ; there is, in fact, a kind of laminar sub-layer. If the particles forming the roughness are completely embedded in this very thin sub-layer, the viscous forces are sufficient to damp out any disturbance created in the flow. When, however, the particles project farther, their disturbances are not so damped, and the turbulence in the outer parts of the boundary layer is augmented, the energy so lost appearing as increased drag. Since the laminar sub-layer decreases in thickness as the Reynolds number increases, we can easily see why a given size of particle becomes more effective in increasing drag as the Reynolds number is raised.

The above remarks apply to a distributed roughness produced by closely-spaced particles, and indicate quite definitely that in the fast aircraft of to-day a surface like the fabric wing coverings of the aeroplanes of yesterday can no longer be tolerated. But there are other kinds of " roughness," such, for instance, as that due to the snap-headed rivets so often used to hold a metal skin to the wing structure, and in this case we may well expect a different effect, since the " particles " are now much larger and are well-rounded in form. A model of such a wing surface was recently made by the Fairey Aviation Co., by pressing indentations into a thin copper sheet, and then wrapping the sheet around a wooden aerofoil. This

has been tested in the compressed-air tunnel and the results are shown in *Fig. 5*. The rivet heads behave like the " carborundum " roughness, in that the drag curve breaks away from that for the smooth wing at a low Reynolds number, but as the Reynolds number is further increased, there is a marked difference of behaviour, the drag reaching a maximum and then falling until at the upper limit it is not much above that of the smooth wing. This behaviour can also be explained. At low Reynolds numbers the flow breaks away

Fig. 5.



Effect of Rivet-Heads on Wing-Drag.

from the rear side of each rivet head and a small region of eddying flow is formed, but at the higher Reynolds numbers the flow succeeds in following the contour of the rivet head and rejoining the aerofoil surface behind it without a breakaway.

I have dwelt at some length on the question of wing drag because of its great importance in relation to modern aircraft design. It goes without saying that such remarks as have been made apply equally to the other surfaces in contact with the air, of which by far the largest is that of the body or fuselage. These also must be made as smooth and free from irregularities as possible if we would attain the highest possible aerodynamic efficiency.

The Maximum Lift of Wings.

We must now turn for a moment to a different kind of aerodynamic phenomenon, but before doing so, a word must be said on an underlying basis of aeroplane design. In order that an aeroplane may take-off and land, it must be capable of being air-borne at a speed low enough to make motion with the wheels in contact with the ground safe. It is this condition which settles the wing area, because a wing gives its maximum lift when set at an angle of some 18 degrees to the wind, and this determines the lowest speed at which the wing lift can balance the weight of the machine. It is therefore of fundamental importance to inquire into the factors which determine the magnitude of the maximum lift of wings, and to see whether means exist by which this lift can be increased. An increase of maximum lift means that the wing area can be decreased for a given landing speed, and this in turn leads to a lower wing drag and a higher top speed. The behaviour of a wing near maximum lift, or, as we usually say, at the stall, is analogous to that of the circular cylinder described in Professor Southwell's lecture. Until this condition is reached the air flows over the wing surface without breaking away from it, but as the lift increases with increasing angle of incidence, so also does the adverse pressure gradient over the front half of the upper surface against which the flow must make its way. There comes a time when the air near the surface has not enough energy to do this, and then the boundary layer separates from the surface, forming a wide wake of eddying flow behind the wing. This is the "stall." The lift ceases to increase, or drops rapidly, depending on whether the separation takes place gently or suddenly. I have not the time to enter deeply into this interesting phenomenon, but those who wish to know more about it will find an excellent account in Professor Melvill Jones's Wilbur Wright lecture of 1934.[1] The only point I wish to make is that any device which reduces the severity of the adverse pressure gradient will help the air to cling to the surface and so promote an increased lift. One such device is the well-known Handley Page slot, in which an auxiliary aerofoil in front produces a down-draught which adds energy to the air flowing over the upper surface of the main wing. A more recent device is the trailing-edge flap so much used on recent aeroplanes, and which is shown in one of its simplest forms in *Fig. 6* (p. 538). The airflow breaks away from the sharp edge of the flap and forms a region of suction behind the flap. The result, as is shown by the pressure-distribution curves in *Fig. 7* (p. 538), is to increase the suction over the

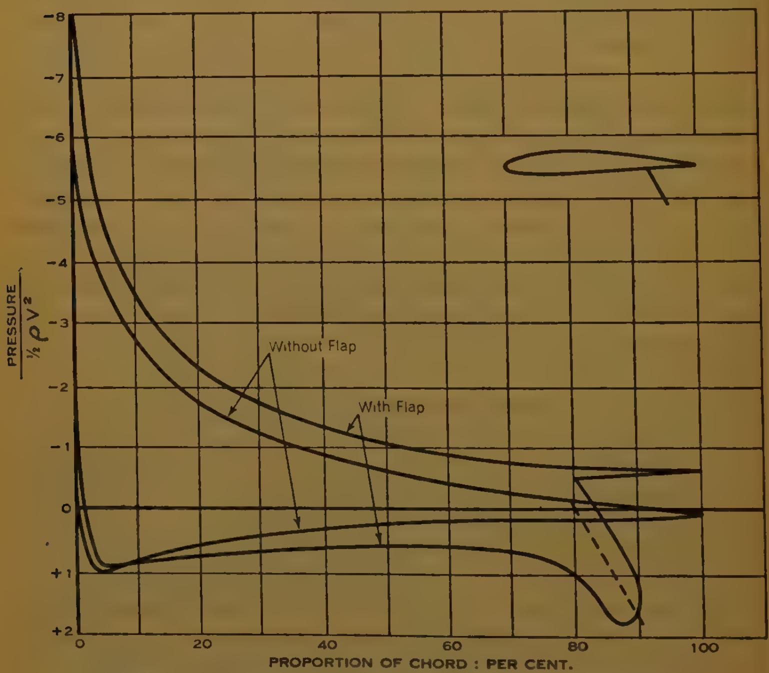
[1] "Stalling," Journal Roy. Aë. Soc., vol. xxxviii (1934), p. 753.

Fig. 6

Aerofoil with Split Flap.

whole of the upper surface of the wing, but without altering greatly the pressure gradient over the front half. The pressure on the under surface, forward of the flap, is also augmented, and there results a large increase of lift. The nature of the lift changes produced by both these devices are shown in *Fig. 8*, from which it will be seen that their behaviour is essentially different in one important practical respect. The Handley Page slot continues the lift curve of the plain

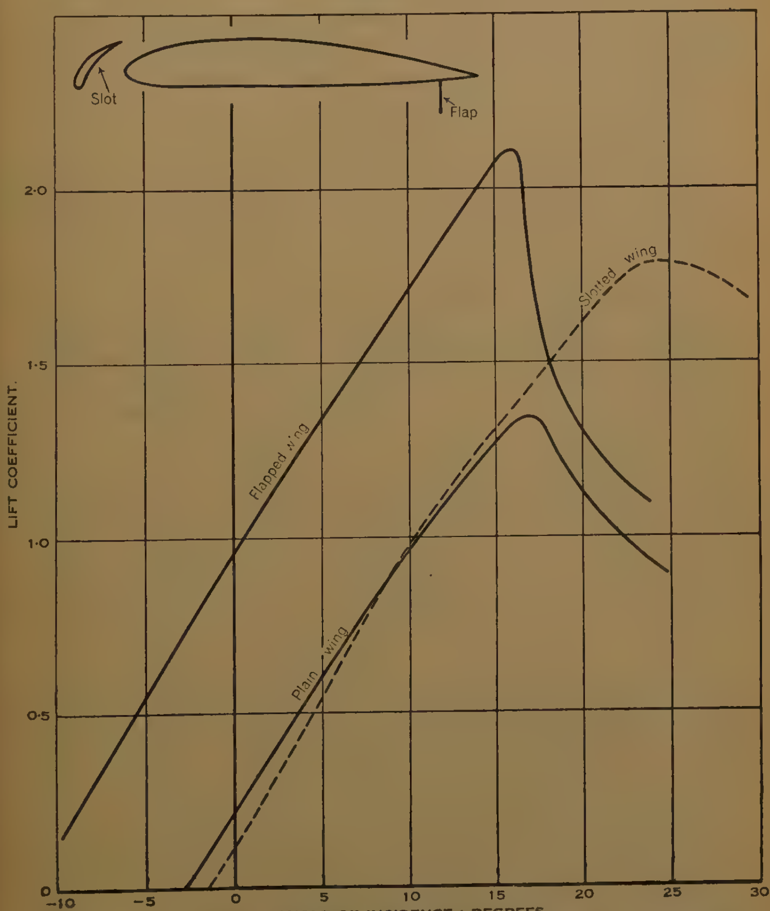
Fig. 7.



Effect of Landing-Flap on the Pressure-Distribution over a Wing.

wing so that it does not stall until a considerably greater angle of incidence is reached, while the trailing-edge flap produces an increase of lift at all incidences, the stall occurring at about the same angle as that for the bare wing. The former device is not very useful as a

Fig. 8.



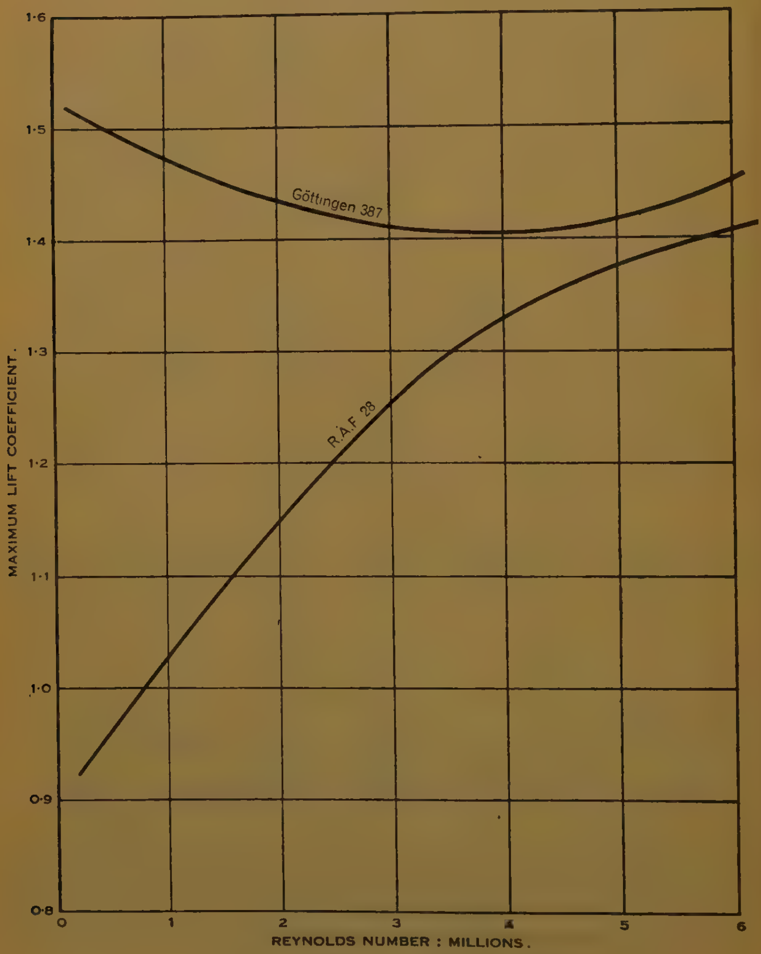
Comparative Effect of Slots and Flaps on the Lift of a Wing.

means of reducing the landing speed, since it involves the use of a very long undercarriage in order to attain the large ground angle necessary to take advantage of the full lift. The flap does not suffer from this disadvantage, and it also possesses other advantages which I shall mention later.

Factors Affecting Maximum Lift.

Much has been said above concerning the scale effect on drag. A few words must be added on the scale effect on lift near the stall, for it is here that ordinary small-scale wind-tunnel tests may be very misleading. The curves of *Fig. 9* show the maximum lift coefficient of two wing sections plotted against the Reynolds number, and the difference of behaviour is at once apparent. The section

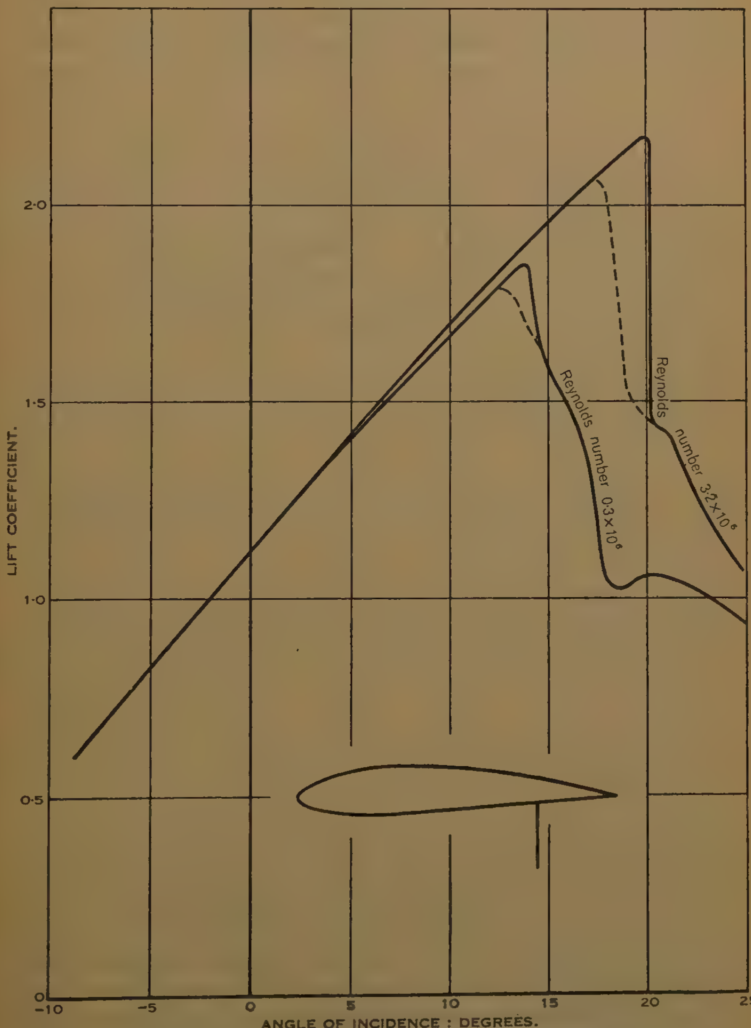
Fig. 9.



Scale-Effect on Maximum Lift.

described as R.A.F. 28 is typical of the moderately thick wings of present-day practice. Its maximum lift increases very considerably

Fig. 10.

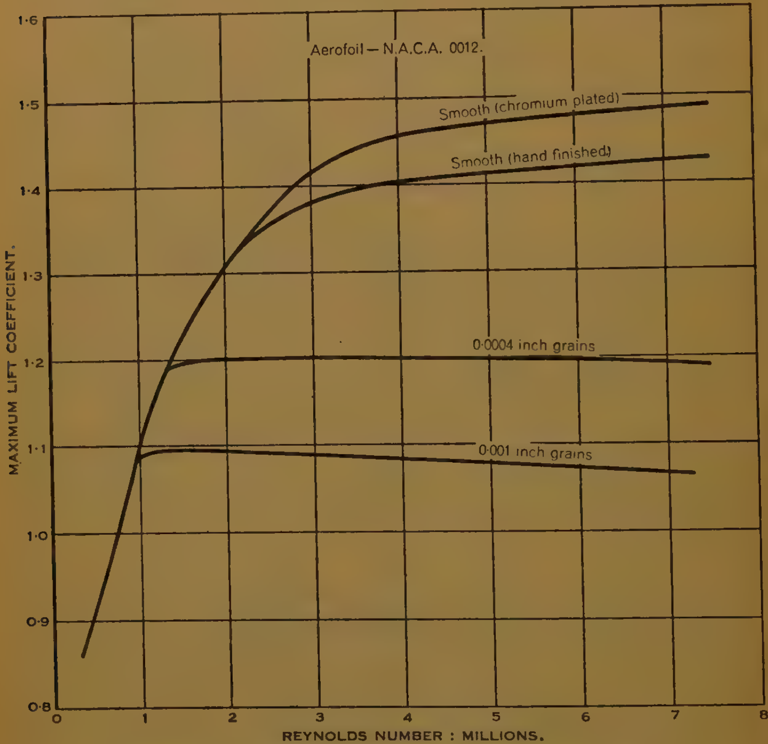


SCALE-EFFECT ON A WING WITH A LANDING-FLAP.

with an increasing Reynolds number. A test in an ordinary tunnel on a model of 6-inch chord at 100 feet per second would give a maximum lift coefficient of 0·95, but at the Reynolds number of a

large aeroplane at landing speed the value approaches 1·4, an increase of some 50 per cent. The other section shown is thicker, and it has a maximum lift which falls with increasing scale. Most thin wings behave in a similar way to R.A.F. 28, and curiously enough all except very thin ones seem to give about the same maximum lift coefficient (1·4) at a Reynolds number of about 5 millions. The curves of *Fig. 10* show that a wing with a landing flap also experiences a

Fig. 11.



Effect of Surface-Roughness on Maximum Lift.

considerable scale effect on maximum lift, the lift rising to a higher value at the higher Reynolds numbers. The dotted part of the curve at high Reynolds number was obtained by decreasing the angle of incidence from a value above the stall, and indicates that near the maximum lift there are two possible regimes of flow, corresponding, in fact, to two different points of breakaway of the boundary layer. This means that in practice the absolute maximum cannot be attained with certainty. Similar dual lift effects near the

maximum have been found in the compressed-air tunnel on certain simple wings without flaps (such as R.A.F. 28) and have been confirmed by flight tests.

Surface roughness has important effects on maximum lift as well as on drag. This is not unexpected, because we know that as a wing approaches maximum lift the flow over the upper surface is in a critical condition and finally breaks away from the surface. The introduction of a further source of energy loss within the boundary layer must modify the critical condition, and an earlier breakaway with a reduction of maximum lift might be anticipated. Recent tests in the compressed-air tunnel show that this is the case. An illustration is given in *Fig. 11*, which shows the maximum lift of an aerofoil (N.A.C.A. 0012) when smooth and when roughened with the carborundum granules previously mentioned. The lift curve for the roughened wing begins to fall below that for the smooth wing at about the same Reynolds number as that at which the minimum drag begins to increase (*Fig. 4*, p. 534) and thereafter there is little further increase of lift, the value at the highest Reynolds number reached being some 20 per cent. less than that of the smooth wing, even with the smaller carborundum grains. When only the downstream half of the wing surface was roughened, the lift curve was the same as that of the smooth wing, but the minimum drag showed an increase of about one-third of that due to roughening all over. It is evident that the forward half of the upper surface is the seat of the instability of flow leading to the stall of this wing, and that any roughness in that locality causes an earlier stall. The drag results indicate that roughness is less objectionable over the rear half of the wing than over the front half, but that the effect is still important.

Flow in the Boundary Layer.

It will have become evident to you by this time that the study of flow in the boundary layer is a most important matter, and that it holds the key to the secrets both of drag and maximum lift phenomena. Many attempts have been made to develop the theory of boundary-layer flow, in order that it may be possible to predict what will happen with bodies of various forms. Mention has already been made of the Blasius solution for the laminar flow over a flat plate. His method can be extended to deal with laminar flow over curved surfaces, and some success has attended attempts to predict the point of separation of the laminar boundary layer from the surface. But this does not carry us very far with practical problems, for in these the Reynolds number is so high that the laminar layer

almost always becomes turbulent before the separation point is reached. Theory has not yet been able to deal adequately with the question of the separation of a turbulent boundary layer: we do not even understand clearly the mechanism by which the onset of boundary-layer turbulence occurs in the simple case of the flat plate, where the question of separation from the surface does not arise. Attempts have been made to discover, from the equations of motion, some sign of instability of flow in the laminar layer, but without success. It appears quite possible that under perfectly steady conditions, as, for example, if a flat plate is moved into perfectly still air, there might be no change to turbulent flow in the boundary layer even at very high Reynolds numbers, but this is at present purely a speculation. In practice there seems to be always a sufficient external disturbance to cause turbulence to appear, but we know that in the case of the flow through pipes, very careful attention to the entry conditions and steadiness of the entering fluid enables a velocity in the pipe far above the normal critical velocity to be reached without the flow becoming turbulent. It must be left to the future to provide the explanation of the phenomena associated with turbulence which we observe, and which are of such importance to the student of aerodynamics; any theoretical advance in this direction will certainly do a great deal to co-ordinate our knowledge of the observed facts.

If good aerodynamic design were no more than to select good wing and body shapes and to make the surfaces smooth, there would be little left but to ensure adequate strength in the structure and to determine the lightest form of smooth skin construction. But the pilot must be able to see clearly when flying, and especially when landing; there must be an undercarriage to land on, and in the case of military machines it must be possible to use guns and bombs. All these considerations introduce modifications which tend to spoil the smooth surface of the machine, and one very important line of *ad hoc* research is to discover how best to obtain the necessary operational facilities without an undue increase of drag. It is hardly necessary to enter into much detail on this very obvious kind of work, but one or two facts that have been discovered deserve a word of mention. One of these is that leakage of air through comparatively small openings in bodies may produce unexpectedly large drag increases. A single opening does not matter so much, but if there are two, and communication is possible between them through the wing or body, then a flow of air occurs, because the external pressures at the two openings are generally unequal. This flow disturbs the boundary layer, and so influences drag. It is not impossible for this effect to be beneficial, especially in the case of

bodies of high drag, but so far as I know, there has been no observed case in which flow through openings in a body of really good form has resulted in a reduction of drag. That considerable increases of drag may result has been abundantly proved by recent tests in the large wind tunnel at the Royal Aircraft Establishment. A second detail that may be mentioned in passing is the design of cabin tops. The open cockpit with its windscreen naturally produces a considerable drag, and in all fast machines of to-day the cockpit is closed by a cabin top having sloping windows for the pilot. It has been shown that there is much to be gained by careful design; in one case the extra drag due to such a structure was reduced to one-quarter by merely rounding off all the corners where the flat panels met. Such work can be carried out fairly effectively in a moderate-sized wind tunnel by using partial models of large scale. The retractable undercarriage is not considered here, as it is primarily a *constructional* and not an *aerodynamic* problem. The only aerodynamic consideration is that the surface should be left as smooth and unbroken as possible when the undercarriage is retracted.

Interference.

Apart from these minor causes of undesirable drag, however, there are two factors of much importance upon which a great deal of research has been carried out, and which cannot yet be considered as completely understood. One of these is the phenomenon generally known as interference; the other is the drag associated with the means adopted for cooling the engine. Interference may be sufficiently well-defined as the effect of putting together two or more components. We may select the best wing for our purpose and make the body a good streamline shape; but what will happen when they are put together, and how does the relative position of the two parts affect the aerodynamic behaviour of the combination? A research into the behaviour of body—wing combinations was inaugurated at the National Physical Laboratory in 1931. A streamline body, similar in shape to an airship, was taken and a wing was added to it in various positions. Tests were also made with the wing above and below the body but not intersecting it. Some conclusions of fundamental importance were soon forthcoming. It was shown that the worst arrangement, from the point of view of drag, was with the body placed on top of the wing and just touching it; the position with the body on the under-surface of the wing was also bad, but not nearly so serious. With the wing well clear of the body, either above or below, the interference was small, as it also was when the wing intersected the body more or less through its centre

line. The explanation of these effects soon followed. I will consider the worst case of the body on top of the wing by way of illustration. It has already been pointed out that the upper surface of the wing is the place where the air finds difficulty in clinging to the surface, owing to the adverse pressure gradient, and that it is here that breakaway eventually occurs at the stall. Put crudely, the air, having passed the thickest part of the wing, has to expand again to fill the space behind, and its behaviour may roughly be compared with the flow through a divergent channel—a conical tube, for instance. If we try to make the air expand too fast it will refuse to follow the surface; the conical tube will not run full, the aerofoil will "stall." Now the effect of putting a body on the top of a wing, especially if the body section is circular, is to produce a pair of rapidly diverging surfaces in the neighbourhood of the trailing edge of the wing, and the result is that the air breaks away from one or both surfaces in this locality and causes an eddying region and an increased drag. The eddying region can easily be discovered by using a short thread of cotton attached to the end of a wire, and exploring near a model in a wind tunnel. Where the air-flow is properly following the surface, the cotton will remain perfectly steady, pointing along the local direction of flow, but in a region of breakaway it will become violently disturbed and may even point in the reverse direction to the general stream. Short pieces of wool, stuck to the surface of an actual machine, were used by Professor Melvill Jones to study the stalling of wings; they may equally well be used to discover sources of interference drag in flight. The consequences of a flow breakaway such as is described above are more serious than a mere increase of drag. In effect, the wing root close to the body has suffered a premature stall, so that lift is reduced also, but still more important is the effect of the eddying flow behind the wing root upon the tail plane and rudder, an effect which may easily upset the control and stability of the machine. It is therefore of great importance to avoid any such breakdown of flow. If a model of a new design is under test, interference effects of this kind will usually be indicated by the drag results. They can then be explored by the use of streamers, and cured by a suitable re-shaping of the surfaces in the affected neighbourhood. If suspected in flight, they can often be explored in the same way.

There is usually a scale effect on this type of interference. Some tests recently made in the compressed-air tunnel showed that the interference drag in two typical cases tended to decrease as the Reynolds number was raised, and this might be expected to occur, since we know that, in general, increase of Reynolds number tends to help the air to cling to the surface. Though wing—body interference is

the most serious form, it will be evident that any place where two surfaces meet at an angle may be the seat of trouble. The junction of tail plane and fin with the body, or of a strut with the wing, must be carefully considered from this point of view. A great deal of experimental data now exist, by means of which the designer may be guided, and there is the general rule always to avoid rapid divergence of two surfaces; it is not, however, easy to give a numerical discriminant whereby any specific case may be judged. An attempt to do this is now being made by the study of flow between divergent surfaces of various forms in the hope that some generalized numerical criterion of breakaway will emerge.

In view of what I have just said on interference, you may be inclined to ask why the low-wing monoplane is so popular, in spite of the fact that it involves a wing—body arrangement which is bad from the point of view of drag. The answer is that by suitably filling in the regions of divergent flow by means of "fillets," a good deal of the bad drag qualities can be eliminated, and that what little inferiority may remain over other arrangements of wings and body is more than compensated by other advantages, of which the greatest is that the close approach of the wing to the ground enables a short, and therefore easily-retracted, undercarriage to be fitted. There is also some gain in ease of landing by reason of the large "cushioning" effect of a wing very close to the ground.

Engine Cooling.

In the earlier days of aeroplane design the engine cooling was achieved without much thought of the resulting drag. Air-cooled engines were placed in front of the body with their cylinders projecting into the air stream, whilst with water-cooled engines a radiator of suitable size was slung under the machine. Perhaps the first attempt to deal with the drag question was the advent of the retractable radiator, which could be drawn up into the body so that less was exposed at the higher speeds where the cooling was more effective. Two simultaneous attempts were made to deal with the radial air-cooled engine. In this country Dr. H. C. H. Townend invented the ring associated with his name, while in America the so-called N.A.C.A. cowling was developed by experiments in a 20-foot wind tunnel. It is not often in these days that a chance observation leads to an important discovery; the Townend ring is a case in point. Townend was considering the possibility of determining the effect of an airscrew slipstream on a body by mounting the airscrew blades radially on the inside of a ring with their inner ends just clear of the body. He made a simple test to see if the ring influenced the body drag and was

surprised to find that in some positions the body actually had a negative drag; it tried to move upstream into the ring. This observation finally led him to try the effect of a ring round the heads of the cylinders of a radial engine, and he found that considerable reduction of overall drag could be obtained. He soon discovered that the action was analogous to that of the Handley Page slot; the ring behaved as an aerofoil whose downwash forced the air on to the body and so prevented the breakaway of flow otherwise produced behind the high-drag cylinders of the engine. The Townend ring is still very widely used. It has the advantage that it can be fitted without interfering much with the accessibility of the engine. The American device took the form of a cowling completely enclosing the engine and extending back to the body behind, there being a comparatively narrow annular slit between it and the body for the escape of the cooling air. Except for detailed improvements the matter remained in this condition until quite recently. Meanwhile the surface radiator for water-cooled engines had been developed, particularly in connection with the Schneider Trophy races. In this radiator a portion of the wing has a double skin, and water is circulated in a thin layer between the two surfaces. In the last Schneider Trophy machines, nearly the whole wing was used in this way, and in addition, other parts of the machine's surface were used for oil coolers of a similar type. The wing radiator seemed to solve the problem of cooling without additional drag, but it had several disadvantages which prevented its general adoption for other than racing aircraft. It was difficult to construct and maintain; it added complexity to the already difficult problem of metal-covered wing construction, and for military aircraft it was far too vulnerable. A development is the use of steam cooling with a radiator, which can now be much smaller, in the leading edge only of the wing, and this scheme has met with some success. As long ago as 1928 Mr. R. McKinnon Wood suggested that the final solution of the cooling problem would be to enclose the engine completely, whether air or water-cooled, and to design passages that would distribute the air to the parts where cooling was necessary. He made some model experiments on a cylinder heated by steam and showed that the cooling could be effected without much drag by a properly designed system of air-ducts.

The advent of the 24-foot wind tunnel at the Royal Aircraft Establishment, a photograph of which is shown in *Fig. 12* (facing p. 531), has recently enabled a more extended research on engine cooling to be initiated in this country, as was done some years back in America, and some very important results have already been obtained. It should be mentioned here that the effect of various forms of cowling

on drag can be determined in a small wind tunnel—the Townend ring was so developed—but the cooling part of the problem cannot be effectively dealt with by small models because the distribution of temperature over the engine is so complex that it cannot be sufficiently well imitated on a model. This is essentially the domain of the large wind tunnel, where a fuselage complete with its actual engine installation and airscrew can be tested under running conditions. By combining such "full-scale" cooling tests with drag experiments both in the large and in the smaller wind tunnels, the staff at the Royal Aircraft Establishment have contributed a great deal to our knowledge of cooling drag and have thrown out the suggestion, paradoxical though it may seem, that cooling may be achieved for less than no drag, or in other words, that the cooling system may actually contribute to the propulsive force provided by the airscrew. Let us examine the basic principles involved. With high-speed aircraft the air is moving too fast for efficient cooling. The cooling effect is roughly proportional to the speed, but the drag of anything used to dissipate the heat varies as the square of the speed. We therefore want low-speed cooling if we would avoid high drag. To attain this, the air must enter some form of cowl in which it is slowed down before it meets the radiator, and this slowing down must be accomplished without loss of energy, that is, without introducing serious eddying motion. Herein lies one of the practical difficulties of the problem, but it does not appear to be insurmountable. Suppose it can be done; we now have a slow-speed stream of air passing through the radiator and so the cooling can be effected efficiently. After the air has passed through the radiator, it must again be speeded up so as to rejoin the external air at a speed not less than the flight speed, again to avoid energy losses. The speeding-up does not present the same difficulty as the slowing-down; it is only necessary to contract the air passage, for, as we saw in considering interference, it is only in an expanding passage that breakaway and eddying occur. But the story does not end here. If the air is slowed down without energy loss, the pressure will rise in accordance with Bernoulli's equation. The heat from the radiator is thus added at a point of high pressure, and the pressure is subsequently reduced as the air is speeded up. The system behaves like a heat-engine, and some of the added heat appears as extra velocity in the issuing air, the increased momentum providing a propulsive force to offset the drag. We have, in fact, a mild form of jet propulsion. As the speed of flight rises, the propulsive effect, like all jet propulsions, becomes more efficient, and may even outweigh the drag due to the skin friction in the air-passages. If it does so, we get our cooling at a negative cost in drag. It is, of

course, necessary also to design the cowl or air passages in such a way that the flow over the outer surfaces of the machine is not spoilt, that is to say, the wings or body must remain properly streamlined externally in spite of the fact that air must be taken in at the front and discharged at the back to provide the internal cooling flow. There are obviously many points of detail needing careful design and experiment before the best can be obtained from such a system, but it certainly appears likely that the cooling of a 300-mile-an-hour machine will soon be achieved for no additional drag, if not for a slight increase of propulsive effect. Contrast this with the figure of 6 to 10 per cent. of the brake horse-power lost in cooling drag on recent machines, and the great importance of this latest work becomes manifest.

The Effect of Increased Knowledge on Design.

I have tried to deal with some of the recent researches related to improvement of aircraft performance, though I have been compelled by lack of time to restrict myself to the more fundamental and striking results. We must now consider how far increased knowledge has influenced design. The Table given early in my lecture has already indicated that this influence has been considerable, and I will now supplement it by showing you a pictorial comparison of the Fairey "Fox" of 1930 (*Fig. 13*), and a modern fighter, the latest Hawker type (*Fig. 14*). The difference of appearance is arresting. The biplane, with its struts and external wiring has given place to the cantilever monoplane in which all the structure is inside the wings and body, leaving the outer surfaces clean and free from any external bracing. The ugly undercarriage, with its unavoidably-poor attempts at streamlining, has disappeared completely. The body, except for the cabin top, has almost as good a shape as an airship. Even the tail plane and vertical fin are unbraced cantilever structures. All external accessories have been eliminated; the wireless generator, for instance, is gear-driven by the engine instead of by a windmill in the air stream. The modern machine is as far removed from that of 1930, in aerodynamic cleanness of design, as was the latter from the earlier "flying Christmas tree" mentioned by Professor Southwell. It is true that we have taken a long time to get to this stage. Professor B. H. Jones pointed out very clearly how far we were from the ideal in a paper published in 1927,[1] and both the Americans and the Germans produced a really streamlined aeroplane

[1] "The Importance of 'Streamlining' in Relation to Performance," Technical Report of the Aeronautical Research Committee, 1927–28, vol. i, R. & M., No. 1115, p. 417.

Fig. 13.

FAIREY "FOX," 1930. *"Flight" copyright.*

Fig. 14.

HAWKER MONOPLANE, 1936. *"Flight" copyright.*

Fig. 15.

DE HAVILLAND "COMET," 1934. *"Flight" copyright.*

Fig. 16.

DE HAVILLAND "COMET," 1934.

before we did. But the characteristic of the Englishman has ever been that when he does realize the position, he wastes no time in catching up and usually passing his rivals. This is well illustrated by the design of the De Havilland " Comet " which won the Australia race in 1934. There were no elaborate wind-tunnel tests on this machine; it was produced simply by the determination of a very able designer to apply all the available ideas on aerodynamic and structural improvement. It was built in an amazingly short space of time, and its success was phenomenal. A side view of the Comet is shown in *Fig. 15*, while the extremely small " frontal area " of the machine is well shown in *Fig. 16*. This aeroplane, and its rival in the race, the Douglas D.C.2 air liner, did much to accelerate the development of the last two years, for they constituted the proof that Professor Jones' ideal low-drag aeroplane was not a figment of the imagination but a practical possibility. I do not wish you to imagine that we have yet reached the ideal; there is still much to be done, but the big step has been taken and there remains only the detailed refinement.

We may give a numerical idea of what has been achieved, and of the margin available for further improvement, by considering a little further the figures given earlier (p. 525), for the Heinkel He 70. This machine is probably the lowest-drag aeroplane yet built and from the German performance figures has a profile drag coefficient of 0·0144. A model of this machine was tested recently in the compressed-air tunnel and the corresponding figure was 0·0151 at a Reynolds number about half that of full-speed flight. As the drag coefficient should be a little lower at the higher Reynolds number, this comparison is very satisfactory. The turbulent skin friction deduced from the flat-plate curve would be 0·0085, and the ratio of the drag of the machine to this is 1·7. From the curves in *Fig. 2* we saw that the wing drag at high Reynolds numbers is somewhat above that of the flat plate, so that we cannot hope to eliminate all the 70 per cent. excess indicated by the above ratio. The radiator drag on the Heinkel was certainly not as low as we now believe to be possible from the arguments I have already outlined. A pure guess is that 40 per cent. of the above 70 per cent. represents a possible future gain, and when this is compared with the improvement of nearly three to one between the Heinkel and the 1930 " Fox," my claim that the big step has been taken appears to be justified.

A word should here be said about the seaplanes which competed so successfully in the last three Schneider Trophy races, and the Italian machine which subsequently won, and still holds, the world's speed record. These machines were the first to be built with

anything approaching the aerodynamic cleanness of design of, say, the Heinkel He 70. It is, however, hardly fair to compare their high-speed performance with that of normal aeroplanes. They were essentially racing machines and every other consideration was subordinated to speed. They carried no "pay load" and only just sufficient fuel for the race, leaving a weight margin that could be used to provide a heavier and more powerful engine. The engine itself was run under conditions that would involve an impossibly-short life for ordinary flight purposes. Most important of all, the wing loading was very high, about 40 lbs. per square foot, involving a landing speed (this was before the days of landing flaps) of about 100 miles per hour, and the machine could accordingly only land safely in ideal weather conditions. These machines did, however, emphasize strikingly the advantages of "clean" design, and formed the starting-point of the campaign for the elimination of parasitic drag which has led to the striking advances of to-day.

While mentioning racing seaplanes, I should add that seaplanes and flying boats have many peculiar problems which do not occur in the design of land machines. The necessity for a good water performance of the hull or floats in landing and taking-off makes it difficult to attain as low a drag in flight as is possible with the fuselage of a land machine. Moreover, either wing-tip floats or stub wings must be fitted to give lateral stability when afloat, and it is by no means so easy to retract these in flight as it is to hide the landing wheels of the aeroplane. Vigorous attempts are now being made to produce seaplanes with something approaching the aerodynamic cleanness of design of the modern aeroplanes, but the problem is essentially more difficult for the reasons just stated, and it seems unlikely that as low an overall drag can be attained. The seaplane has, however, the advantage that it can land safely at a considerably higher speed, and this enables a certain measure of the high-speed performance to be regained in spite of the higher drag.

Structural Considerations.

A great deal of the aerodynamic improvement above outlined has been rendered possible by developments in other fields. An enormous amount of research and ingenuity of design has been occupied with the problem of new materials and their efficient application to the structures involved in the modern aeroplane. The metallurgist has provided the constructor with better and better steels and light alloys, whilst research on the behaviour of structures built up of thin sheets and strips has shown how to use them to the best advantage. The problems are essentially different from and far

more difficult than those of heavy engineering, where the designer has an ample factor of safety and is usually only interested in the strength of components in direct tension and compression. In the case of the light thin-material constructions to which the aircraft designer is forced, failure is liable to occur due to the elastic instability, or buckling, of some member before the maximum allowable direct stress is reached, and the problem is to design so that the full stress can be attained without any failure by buckling. The necessity to save weight leads further to an attempt to make every possible piece of material contribute its full quota to the strength and stiffness of the machine. We may take the wings as an example. In earlier days the strength resided in the spars and the external bracing. The ribs were necessary to hold the fabric covering to the correct shape, but neither they nor the fabric contributed very much to the strength. The strength calculations were easy, for the structure was no more than an ordinary braced girder. In the modern wing a smooth surface is essential, as we have seen, and one of the obvious ways to attain it is to use a sheet metal skin. But this is heavy, since there is a limit to the thinness of sheet that can be satisfactorily held to shape with a reasonable number of supporting frames. Hence the metal skin must take its share of the stresses if it is to be economically possible, and there results a structure in which the main spars, the ribs, and the covering are all integral parts of the stressed system. Strength calculation becomes much more difficult, for not only is the system one with many redundancies, but, as stated above, failure by elastic instability must be considered in many components as well as failure under direct stress. But this is by no means the end of the story; there still remain a number of compromises to be made between weight and aerodynamic considerations. From the point of view of strength as a cantilever, the tendency is towards a thick wing section at the root and a wing highly tapered in plan form. But if the thickness of a wing exceeds about 15 per cent. of the chord, the profile drag begins to increase fairly rapidly, and so a compromise arises between drag and wing weight. A large degree of taper leads to a large chord near the wing roots in order to provide the necessary total area. This in turn leads to larger pitching moments due to movement of the centre of pressure of the wings with change of incidence, and so necessitates a larger tail area for stability. The designer must endeavour to make the best compromise between these and other conflicting factors, and at the same time keep a wary eye on a number of new factors which are related to the *stiffness* of the structure as distinct from its *strength*. Professor Southwell instanced the problem of wing flutter as a case where complex mathematical and experimental investigation had done

much to eliminate a danger first made apparent by increasing speed. The remedial measures suggested by the investigation were widely applied and as a result flutter accidents became rare. The recent great increases in flying speed have, however, made further consideration of flutter necessary, not because the preventive measures are now inadequate but because of the difficulty of applying them with sufficient precision to ensure absolute safety at the higher speeds. A desirable feature is a wing very stiff in torsion, and this gives a great advantage to the stressed-skin wing, but often at a cost of more weight than is necessary from the pure strength point of view. There is also another consideration which demands great torsional rigidity of the wings, and that is the efficiency of the aileron control. If the wing is too weak in torsion, the application of the ailerons twists the wings to such an extent that the rolling moment due to the ailerons is largely offset by the moment of opposite sign due to the twist of the wing. In an extreme case the rolling moment on the machine may actually be reversed, so that use of the ailerons will roll the machine in the opposite direction to that expected. What is needed here, as also for the prevention of flutter, is a type of aileron which applies a rolling moment to the wing without twisting it, or, in other words, an aileron which applies an increment of lift at the centre of pressure of the wing instead of near the trailing edge. No completely satisfactory device of this kind has yet appeared.

Similar considerations of rigidity apply also to the fuselage, the torsional rigidity of which must be high in order to minimize the possibility of flutter of the tail surfaces at high speeds. The result of such considerations in practice is a tendency towards the use of all-metal construction in which both wings and fuselage are stressed-skin structures. The structural problems are quite different for the thin wing bearing the lift and for the almost circular fuselage bearing little direct aerodynamic load, and much research has been directed to the best use of material in the two cases. The so-called " geodetic" structure developed by Mr. B. N. Wallis, M. Inst. C.E., of Messrs. Vickers Aviation Co., is particularly well adapted to give a light fuselage very rigid in torsion, and has also been applied successfully in wing construction. Here, curiously enough, it leads us back in one way to the older form of wing, for as the strength and rigidity lies essentially in the framework, it is desirable to use the lightest possible unstressed wing covering.

The modern metal-covered wings are much heavier than the older wood and fabric type, but they are not necessarily heavier in relation to the weight of the aeroplane. The old type weighed about 1 lb. per square foot, and was used on machines with a wing loading of from 6 to 10 lbs. per square foot. The metal wing may be between

2 and 3 lbs. per square foot in weight or even higher, but the loading of the modern machine is between 20 and 30 lbs. so that the relation of wing weight to total weight is almost unaltered. The rigidity of the metal construction is enormously higher, so that there has been a gain in the direction of safety from flutter and like troubles without a sacrifice of percentage wing weight. Fabric coverings are, however, still largely used, especially when extreme stiffness is not important, or where, as in the " geodetic " wing, it is obtained in another way. Attention is being given to processes for obtaining a really smooth finish on fabric, and means have been devised whereby the string used to sew the fabric to the ribs can be embedded so that it does not form projections as in the older form of sewing. When such precautions are taken, there seems no reason why a sufficient smoothness should not be obtained to render the drag of a fabric-covered wing as low as that of the metal-covered one.

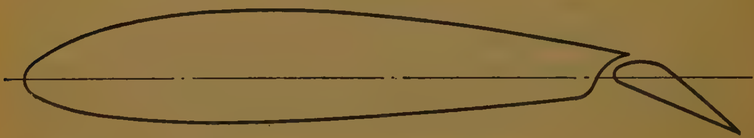
The Taking-Off and Landing of Aeroplanes.

I must now leave the question of high-speed performance, and consider briefly other matters concerning the operation of aircraft. I have already mentioned the landing problem in my remarks on high-lift devices, but there is much more in it than the mere reduction of landing speed by means of such devices. Strange as it may seem at first, the better an aeroplane is made from the point of view of low drag and consequent high speed, the more difficult it becomes to land. The reason for this is two-fold. In the first place the gliding angle is reduced by the low drag, so that the machine comes in to land on a less inclined flight path, and so must travel farther after clearing obstacles near the aerodrome before it touches the ground. Secondly, the low drag means less resisting force from the air during the run on the ground, which is therefore lengthened. The aeroplane of 10 years ago was easy to land on account of its high drag. The problem to-day is to make the efficient modern machine temporarily as inefficient as that of a decade ago during the landing operation. It is here that trailing-edge flaps are very effective, for not only do they decrease landing speed by increasing the maximum lift, but they also increase the drag very considerably. Nearly all modern heavily-loaded aircraft are fitted with such flaps, differing in some respects in design, but all operating on the principle previously mentioned, namely, to produce a suction effect behind the flap which augments lift and drag simultaneously. The flaps have so far been capable only of comparatively slow opening by means of worm gear or small hydraulic rams, the method of use being to set the flap at a suitable angle before commencing the glide, and leaving it there

during the whole landing. It will be realized that the forces on such flaps, especially on a large machine, are very considerable, and that it would be no easy matter to balance them aerodynamically so that they could be operated quickly. Nevertheless, attempts are now being made to do this, for it is felt that if they could be used as a control during landing, they might make the landing operation of a heavily-loaded machine easier to perform. Some idea of the very large effects produced by flaps may be gained from the fact that it is easy, by their use, to halve the distance to come to rest from a glide starting at a height of 50 feet.

Another problem of ever-increasing difficulty which confronts the designer is that of "take-off." Here again flaps may be of some assistance, but they must be used differently. The high drag is now not required; it is in fact a hindrance to the rapid acceleration required in the take-off run. Increase of lift, however, is advantageous, for it reduces the speed at which the machine becomes

Fig. 17



HANDLEY PAGE SLOTTED FLAP.

airborne. The ordinary split flap, shown in *Fig. 6* (p. 538) will help take-off if put down slightly, say to about 15 degrees, for it then gives an appreciable extra lift without much increase of drag. The slotted flap developed by Messrs. Handley Page (*Fig. 17*) is better, for it gives a greater extra lift with very little change of drag when put at a small angle, and is still as effective as the split flap for landing when put down to 60 degrees or so. The question of take-off is, however, mainly a matter of airscrew performance. As the speed of aircraft has increased, it has been necessary to use airscrews of higher pitch in order to get the requisite thrust at the higher forward speeds. The blades of an airscrew behave, as is now widely known, very much like the wings of an aeroplane, giving a "lift" which is the propulsive thrust, and a "drag" which provides the reaction to the engine torque. For maximum efficiency the angle which the blades make with the relative wind, compounded of the forward motion and the rotation of the airscrew, must not be too great; it is analogous to the angle of maximum lift/drag ratio for the wing. The blade angles along the radius of the airscrew are so chosen that each section works near its maximum efficiency under the conditions

of normal flight. Now when the aeroplane is at rest or moving slowly during the take-off run, the angle at which the airscrew blades meet the relative wind is increased, and if it is sufficiently increased the blades will stall just as a wing does at a large angle of incidence. The greater the difference between take-off and top speed, the greater will be this tendency of the blades to stall. If the blades are stalled, their "lift" is reduced, or, put in airscrew terms, the thrust is reduced. With the high-pitch airscrews used on modern fast aeroplanes, the airscrew blades may be stalled during the whole of the take-off run, and the consequent loss of thrust may make the run very much longer than it would be if the thrust could be maintained at its maximum value with the blades unstalled.

It is on account of this behaviour that the two-pitch or variable-pitch airscrews are finding more and more favour. In these airscrews it is possible to alter the blade angles by a mechanism which rotates the blades about an axis along their length. Even this does not completely solve the problem, for at the lower pitch setting for take-off the torque is lower, and it may not be possible to absorb the full power of the engine at its highest permissible rate of revolution. This could be done if a two-speed gear box were fitted as well as a variable-pitch airscrew, but you will readily appreciate that this involves a good deal of complexity and additional weight. The variable-pitch airscrew, with metal blades, weighs nearly three times as much as a fixed-pitch wooden screw, and the extra weight must be carried during the whole flight of the aeroplane although it is only actually needed during the 15 or 20 seconds of the take-off run. It is therefore no wonder that consideration is being given to the possibility of using other means of providing additional external power during the take-off, such as by a catapult. An *assisted* take-off of this kind has the effect of making it possible to overload the machine very considerably, provided that the overload is lost before the machine has to land. Two obviously important applications of this possibility are to the long-range commercial air-liner where the overload can take the form of extra fuel, and to the bombing machine where it takes the form of extra weight of bombs.

Stability and Control.

When the aircraft is in the air, there arise all the problems of stability and control, and the latter are essentially different for the commercial aeroplane, which flies at a constant cruising speed in a straight line, and for military aircraft, which must be capable of rapid and violent manœuvres. Professor Southwell, in his 1930 lecture, told you how the problem of stability of flight had been

dealt with mathematically and experimentally. Unfortunately the analysis is so complex that there is little hope that it will ever be applied fully in the design office. The aeroplane has six degrees of freedom; it can move linearly along three directions, mutually at right angles to one another, and it can rotate about three similar axes. The forces acting upon it can be resolved into three components of force and three components of moment. To consider the most general case of stability of motion, we must know the rate of variation of all six forces and moments with all six linear and angular velocities, thirty-six quantities, or "derivatives," as they are generally called. Some of these are easy to calculate or measure; some very difficult to determine. The complexity of a complete treatment of stability will have become very evident to you. What we try to do is to use stability theory and numerical calculations, based on typical values of the derivatives, to show us in what direction we must modify a machine in order to eliminate any particular form of instability which it may exhibit. Such calculations tell us, for instance, that a tendency to diverge slowly from straight flight may be cured by a reduction of fin area, and that a lateral oscillation involving turning and rolling is due either to too little fin or too much dihedral angle on the wings. In practice, experience soon leads to types of aircraft which are satisfactory as regards stability, and as long as only small variations in general design are made, there is not likely to be any serious difficulty. If, however, a radical change is made, as has in fact actually been done recently in the general adoption of the monoplane, then experience may sometimes fail to be a sufficient guide. Cases have recently occurred in which the stability of some of the modern monoplanes has not been satisfactory, because the rules developed from experience of the thin-wing biplane cannot be applied sufficiently well to the new type, and also because there exists comparatively little experimental data on the stability derivatives of the monoplane in comparison with the vast collection of such data for the older types of machine. Research is in hand to remedy this state of affairs, and to investigate fully the stability of typical modern machines. There is little likelihood, with the knowledge we possess, that dangerous instability will occur with a new design, but a great deal of time may be lost if new machines have to be modified after completion in order to cure them of minor instability. An unstable machine can be flown safely, but the process is tiring to the pilot by reason of the constant demand on him for slight correction by the controls, and no machine can be considered really satisfactory unless it can be flown "hands off" over at any rate the greater part of its speed range.

The provision of ample control by the ailerons, elevators, and rudder does not present any difficulty in normal flight, and the only serious problem here is the balancing of the control surfaces, particularly on large machines, so that they may be easily operated without undue fatigue to the pilot. It is evident that, if aeroplanes increase in size, there must come a time when this is no longer possible, and where some form of servo-mechanism must be employed to move the controls, just as such means have long ago been necessary to move the rudders of large ships. Servo-controls have already been used on large aircraft, and they raise a number of very difficult problems relating to balance and also to the possibility of flutter. The modern tendency is to avoid servo-control if possible, and to rely on as delicate an aerodynamic balance as can reasonably be attained, coupled with so-called " trimming " devices which enable the pilot to reduce the hinge moment on a control to zero for any steady condition of flight so that in maintaining that condition he is only called upon to make small control movements involving little effort. These " trimmers," or " tabs," usually take the form of very narrow hinged strips on the trailing edges of rudder and elevators, operated by a separate lever or wheel under the pilot's control. Applied to the elevators, they have now almost completely replaced the adjustable tail-plane of earlier machines as a means of trimming to any desired speed of flight. In this connection they help clean design, because the tail plane can now be made an integral part of the fuselage, and the awkward gaps associated with an adjustable tail plane can be eliminated.

Control at the stall is still a matter that is engaging much attention. It is primarily a matter of safety. In ordinary use a machine is not flown at so low a speed that difficulties of control associated with the stall are likely to occur, but in landing it is desirable to approach the ground at as low a speed as possible consistent with a sufficient margin for the " flatten out " just before touching the ground. If the lateral stability and control are bad under these conditions there is a risk, especially in bad weather, of a sudden dropping of a wing, and the consequences may be serious on account of insufficient height for recovery. The use of automatic slots at the wing tips, as developed by Mr. Handley Page, did a great deal to avoid this danger, and they have been extensively used. There is, however, a natural desire to avoid the use of such a device, if possible, for it adds mechanical complexity and also makes it more difficult to secure a perfectly smooth wing surface when the slots are shut.

From time to time unslotted machines have appeared which were reported to be as good as slotted ones with regard to lateral control in stalled flight, and to-day there is a definite tendency not to use

tip slots. The whole situation is complicated by the increasing use of wings tapered in plan form, the behaviour of such wings at the stall being different from that of the wings of uniform chord used on the older machines. The parallel wing stalls first at the centre of its span, the stall spreading outwards as the angle of incidence is further increased, so that the tips remain unstalled for a while and provide a certain amount of damping to a rolling motion. A very highly tapered wing stalls first at the tips, the stall spreading progressively inwards to the centre. At a certain value of the taper, about 2 to 1, the tendency is for the whole wing to stall at once. The Prandtl theory shows that a wing of elliptic plan form should stall simultaneously at all points of the span, and the 2-to-1 straight taper gives a shape not greatly different from the ellipse. One would expect the elliptic wing, or the equivalent tapered wing, to be the most critical as regards tendency to auto-rotate when at the stalling angle, and therefore the most likely to cause difficulty in control at low speeds. This conclusion is borne out by model experiments and by flight experience. Unfortunately, the critical taper of about 2 to 1 is very convenient from the constructional point of view, and we are again faced with a compromise between two conflicting requirements. A likely solution is a reversion to the use of tip slots to prevent early wing-tip stalling. The addition of landing flaps further complicates the whole problem, for these flaps, by giving a concentration of lift at the centre of the span, produce an additional "upwash" near the wing tips tending to promote earlier stalling of the tips. With flaps extending over the inner half of the span, the effect is not serious, and there is no reason to anticipate any greater difficulty in slow-speed control than would occur with the flaps out of use. If, however, it is desired to use landing flaps over the whole wing span in order to retain a reasonable landing speed with a much higher wing loading, a very difficult problem arises which has not yet been satisfactorily solved. Not only does the tendency to rolling instability become much more marked with the full-span flaps, but the aileron control is seriously reduced by the presence of the flaps in front of the ailerons. Attempts are being made to devise some form of rolling control that can be used effectively with full-span flaps. At the moment the commonest design is one with half-span landing flaps and conventional ailerons, and this provides sufficient "flap" effect for the loadings now in use, say up to 25 lbs. per square foot. The future may force us to higher loadings, and then the use of full-span flaps for landing will become a necessity, and the accompanying difficulties of control will have to be solved.

THE PROBLEM OF FLUTTER.

I have already referred briefly to the problem of flutter, and to the likelihood that the great increase of speed of machines now being built and the still higher speeds of the future will again bring it into prominence. The theory has been very fully worked out for wings and tail surfaces on the assumption that elastic and aerodynamic forces can be considered as linear functions of the displacements, these being the usual assumptions in the treatment of the stability of small oscillations. Attempts are now being made to extend the theory beyond these limitations. In practice some of the aerodynamic quantities involved, such as the hinge moment on a control surface, are not linear functions of displacement, and there are other departures from linearity such as the existence of solid friction in the moving parts. It has already been shown that such non-linearities can lead to flutter at a lower speed than the critical value given by the small-oscillation theory, provided that a sufficiently large initial disturbance is imposed. It is accordingly desirable to study the general problem of the oscillations of dynamical systems obeying non-linear laws, and to apply the knowledge so gained to the flutter problems that arise in high-speed aircraft. There is here an opportunity for the application of mathematical research to a practical problem. So far, the study of such systems has been very limited, being confined, as far as I know, to certain cases of oscillations in electrical circuits. The mathematics will certainly be difficult, and it will probably be necessary to use mechanical aids for the solution of the equations. The possibilities of such machines as the "differential analyser" devised by Dr. V. Bush in America are being explored in this connection. A suitable machine may be able to solve in half an hour an equation which would take weeks of step-by-step calculations to evaluate. In fact, the lengthy calculations involved in the application of flutter theory are, at the moment, the greatest bar to the rapid extension of knowledge of flutter phenomena. It is interesting to note, in passing, that in many branches of science equations arise which cannot be solved except by lengthy numerical processes, and that there is a definite indication that the future will see a great extension in the use of machines for the solution of such equations. It is perhaps not too much to hope that even the intractable equations of viscous flow may one day yield to such treatment.

Turbulence.

I can hardly leave the subject of aerodynamic research without some mention of the investigations at present being carried out upon the nature of turbulence. At the moment these can only be classified as abstract science, and we cannot yet see their application to any practical problem of aeronautical engineering. I have already indicated that most of the aerodynamic phenomena which make flight possible are intimately connected with the behaviour of the air close to the surface of bodies, and that the onset of turbulence in the "boundary layer" is of fundamental importance in determining the resultant forces on the bodies. In order to understand these phenomena completely it is obviously necessary to know the precise nature of the turbulence which occurs, and many experiments have recently been devised to increase our knowledge in this direction. Hot-wire anemometry has been employed to give records of the velocity fluctuations in turbulent flow; the ultramicroscope has been used to watch and photograph the turbulent movements in free streams and close to surfaces; and a method depending on the photography of the motion of minute spots of hot air produced by tiny electric sparks has been employed to follow the details of the turbulent motion. Meanwhile, Professor G. I. Taylor, at Cambridge, has done a great deal to advance the statistical theory of turbulence, and his work has suggested a number of experiments. One of the difficulties of such experimental work lies in the fact that there is always initial turbulence in the air-stream of a wind tunnel, and this, as we have already seen, complicates the interpretation of any experiments made on turbulence in the boundary layer. At the National Physical Laboratory we are at present designing a wind tunnel in which the flow shall be as free as possible from turbulence, and with means of creating regular turbulence of varying intensity as desired. With such a tunnel many investigations which are now difficult or impossible could be carried out, and would undoubtedly do much to advance knowledge of the subject. I have not time to describe the very interesting results that have already been obtained in such investigations: I can only instance this line of research as one that may have a profound influence in the future.

The Future of the Airship.

Before I attempt to summarize the present position and to venture a prophecy for the future, I feel I should say a word or two on the airship, a subject on which Professor Southwell spoke at some length in 1930. There is here little progress to report, for the un-

fortunate disaster to R.101, followed by the American accidents, practically put a stop to airship development in both countries. Whether the airship will ever come into its own as a means of long-distance passenger transport is a question on which it is difficult to form an opinion. It certainly offers a more comfortable and quiet means of air travel than at present seems possible with heavier-than-air craft, and at a speed which, although low compared with that of present-day aeroplanes, is still high compared with that of ocean-going ships. The performance of the "Graf Zeppelin", which has now crossed the South Atlantic some 112 times, shows that it would be unwise to suggest that the airship has no future. The German authorities evidently think otherwise, and the career of the larger airship they have just completed, which has a reputed range of some 8,000 miles, will be watched with much interest. I can only add that in the event of a resuscitation of the airship in this country, our present increased knowledge of skin-friction drag, coupled with the past work on the strength of airship structures for which Professor Southwell was so largely responsible, should place us in a position to improve very considerably upon past efforts.

Summing-up and Future Progress.

I feel that I have failed to mention many things that are of importance in the development of modern aircraft design; I can only plead that a lecture must be of finite length, and that I have tried to select those points which seemed to me to have been the most potent causes of the recent accelerated improvement of aircraft performance. I have, for example, said nothing of engine development, but I do not wish you to assume that nothing has been done in the period of my review. Very great efforts have continually been made to improve the efficiency and reliability of the petrol engine and to develop other types, such as the compression-ignition engine, for aeronautical purposes. But I contend that my broad statement early in the lecture is correct, and that as the period of Professor Southwell's review was primarily one of great improvement in the power unit, so the period I have dealt with was primarily one of increasing aerodynamic efficiency. To sum up as best I may: there has been a "cleaning up" of the aeroplane unprecedented in its history, the modern machine has been reduced almost to the bare essentials of the wing to lift it and the body to hold its contents, all extraneous drag-producing accessories have been removed or placed inside, the surfaces of wings and body have been made areodynamically smooth, and structural design has tended towards the

use of thin metal placed as near the surface as possible so as to secure the maximum stiffness with the least weight.

Only a few machines have so far been built incorporating all these improvements, but the very near future will undoubtedly witness their almost universal application. Can we see any hope of future progress on these lines ? There are three ways of progress ; further reduction of drag, improvement of the engine, and reduction of structure weight. Will abstract science again, as in the past, provide new discoveries that will produce as great a change as that which has occurred in the period with which I have dealt ? There will doubtless be steady, slow progress along all three avenues, but it would appear that nothing short of a revolutionary discovery can produce another spectacular change. We have seen that the margin remaining over pure skin friction drag is only 70 per cent., and I have hazarded a guess that perhaps 40 per cent. of this may be gained by persistent attention to detail in design. A certain amount may also be gained by the reduction of surface other than the lifting surface of the wings, that is, by a closer approach to the so-called "flying wing." With small machines the effect of the surface reduction would probably be counterbalanced by the higher drag of the very thick wing that would be needed, but with large machines a considerable gain may ultimately prove possible in this direction. The revolutionary discovery we need as regards surface friction is to find a means to make the boundary-layer flow remain laminar over a much greater proportion of the surface, or in other words to prevent turbulence from developing in the boundary layer itself. If this were possible, the drag of bodies at very high Reynolds numbers would be reduced to about 10 per cent. of the values to which we are accustomed. The prospect appears so bright that one is inclined to assume *a priori* that there must be a physical impossibility in its achievement, but I do not believe it is possible to prove that this is the case. If our researches on the nature of turbulence were to enable us merely to delay the inception of turbulence until say half the exposed surface has been passed, the resulting drag reduction would be very considerable.

Great improvements in structure weight and in the power unit seem again only possible as a result of some new fundamental discovery. It is not impossible that research in atomic physics will open the way to the production of synthetic materials hitherto undreamed of, and that these will react greatly upon aircraft design. Still more speculative is the possibility that we may one day find a lighter means of producing or storing power than is provided by the heat-engine and its fuel. Even if such discoveries are eventually made, it is doubtful if speeds of flight will ever rise very greatly,

although we could, of course, obtain present speeds much more economically and also increase the range of aircraft enormously. My reason for saying that very great speed increases would not be made is twofold. In the first place, the effect of compressibility of the air on drag sets a serious limit to speed, for as the speed of sound in air is approached, the drag rises very rapidly indeed. When we realize that the world's speed record is already nearly six-tenths of the speed of sound, we see that compressibility effects will become of paramount importance if great increase of speed is rendered possible by new discoveries. In the second place, there is the question of the temperature attained by a body moving rapidly in the air. The air is brought to rest on the surface and the temperature rise is of the order of the adiabatic change associated with the velocity. At 600 miles per hour the adiabatic rise is 36° C. and it varies as the square of the speed. The consequences of this upon the problem of engine cooling, to say nothing of cooling the passengers, is obvious. Both these difficulties can be overcome by flying very high, the drag being reduced because of the lower density, and the temperature rise being then even desirable on account of the low external temperature, but very high flying brings its own problems of maintenance of engine power in air of low density.

With these somewhat wild speculations, I will conclude my lecture. I trust that I have been able to give you some idea of the recent improvements in aircraft that have been effected, and to trace the connection between research and practice. If I have succeeded in doing that, I shall feel that I have added something of value to the objective which is the basis of this series of lectures.

I cannot close without acknowledging the help that I have received from various quarters in the preparation of my lecture, and especially the permission of the Air Ministry to use some of the illustrations and numerical data, the kindness of the Hawker, De Havilland, and Fairey Aviation Companies in providing me with illustrations, and the permission of the Editor of "Flight" to show some of the photographs of machines, of which he holds the copyright.

Sir Robert Hadfield, in moving a vote of thanks to the lecturer, said that those present had heard a very valuable address indeed, and that it was most fortunate for The Institution that Mr. Relf had joined the ranks of the Forrest Lecturers. He was sure that James Forrest would have been exceedingly pleased had he known that such an admirable lecture would be forthcoming, presenting a most abstruse subject in a very able manner.

He noticed that Mr. Relf had referred to stratosphere flying, and thought that that was where he would have to ask the

metallurgist to come in and help him. Unfortunately, beyond a certain low temperature iron became brittle, and there were not many ferrous alloys which had a proper toughness and ductility at the low temperatures which prevailed in the stratosphere. No doubt Mr. Relf would study that point, as it would certainly have to be faced. In his own Forrest Lecture on "The Unsolved Problems of Metallurgy" [1] he had looked forward a little, stating that for the progress of engineering development it was essential to have steel of the very highest quality, particularly in regard to toughness. He thought that British metallurgists had done their best to try to help the aeroplane builder by finding steels which could be relied upon. In a recent paper before the Royal Society [2] he had described how the metallurgist had now produced steel containing 57·5 per cent. of nickel which, notwithstanding the exceedingly low temperature of liquid hydrogen (—252·8° C.), gave a tenacity of 48 tons per square inch with a ball hardness (estimated) of about 346 and, on return to normal temperature, 187. The aeronautical engineer thus had reason to hope that he would in the future find the materials that he would require.

Mr. F. E. Wentworth-Sheilds, in seconding the vote of thanks, said that he was sure that all present had listened with intense delight to the Lecture. It had again impressed upon them that research work was bound to be essential to the engineer. The Institution had recently in many ways given special encouragement to research workers because it felt that the practising engineer was becoming more and more dependent upon them, and had cause to be grateful to them at every turn for the work they did. He felt sure that that feeling was very strong among those who had heard Mr. Relf's admirable description of recent research on aircraft.

The motion having been carried by acclamation,

Mr. Relf, in acknowledging the vote of thanks, said that it had been a great pleasure to him to be associated with the James Forrest Lectures.

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[1] Minutes of Proceedings Inst. C.E., vol. clxvi (1905–6, Part IV), p. 190.

[2] Prof. W. J. De Haas and Sir Robert Hadfield, "On the Effect of the Temperature of Liquid Hydrogen (— 252·8° C.) on the Tensile Properties of Forty-one Specimens of Metals comprising (*a*) pure iron 99·85 per cent.; (*b*) four carbon steels; (*c*) thirty alloy steels; (*d*) copper and nickel; (*e*) four non-ferrous alloys." *Phil. Trans.*, vol. 232 (1934), p. 297.

ANNUAL GENERAL MEETING.

12 May, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

The Notice convening the Meeting was taken as read, as well as the Minutes of the Annual General Meeting of 14 May, 1935, which the Chairman was authorized to sign.

The following Report of the Council (p. 570) upon the Proceedings of The Institution during the Session 1935–36 was read, the Statement of Accounts (pp. 588–598) being taken as read.

The PRESIDENT moved—That the Report of the Council be received and approved and that it be printed in the Journal of The Institution.

Sir ALEXANDER GIBB seconded the motion.

Mr. R. H. THORPE, in discussing the motion, referred to the activities of the Local Associations of Students and Corporate Members, at the birth of the earliest of which he had been present. He described the work of the Students' Committee in 1883, when Mr. Sidney Locock had been chairman, and in 1884 and 1885 when he had been chairman, and mentioned the steps taken to obtain Papers from other parts of England in addition to those from London. The initiative had been taken by the Students themselves, and it was only later that the Council had further developed the idea. In connection with the possibility of closer co-operation with other Institutions, he recalled that in 1879 Sir William Siemens had offered to give £10,000 as the nucleus of a fund to bring the Institutions of Civil and Mechanical Engineers together in one building; that offer was, however, not accepted. He mentioned the necessity for bringing the Benevolent Fund to the notice of members as prominently as possible. With regard to the election of the Council, the alphabetical balloting list employed was unsatisfactory in that members whose names came towards the end of the alphabet were not so favourably placed as others; also, he understood that few ballot-papers were received from members overseas. He thought that the method of voting might well be improved.

Mr. MEASHAM LEA expressed his approval of the new form of publication of the Proceedings, and said that the Journal was more readable and up-to-date than the "Minutes of Proceedings."

Mr. T. H. Webster suggested that copies of the Papers read at the Meetings should be more conveniently placed for the use of visitors and others.

The Meeting then resolved—That the Report of the Council be received and approved, and that it be printed in the Journal of The Institution.

The Scrutineers reported the election of the Council for 1936–1937, as follows :—[1]

President.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S.

Vice-Presidents.

Professor William Ernest Dalby,[2] M.A., F.R.S.
Sydney Bryan Donkin.
Sir Robert Abbott Hadfield, Bart., D.Sc., D.Met., F.R.S.
William James Eames Binnie, M.A.

Other Members of Council.

Athol Lancelot Anderson, C.B.
David Anderson, LL.D., B.Sc.
Thomas Henry Bailey.
Kenneth Alfred Wolfe Barry,[2] O.B.E.
Asa Binns.
John Job Crew Bradfield, C.M.G., D.Sc., M.E.
Raymond Carpmael, O.B.E.
Frederick Charles Cook, D.S.O., M.C.
Sir Harley Hugh Dalrymple-Hay.
Jonathan Roberts Davidson, C.M.G., M.Sc.
Sir Eustace Henry Tennyson d'Eyncourt, Bart., K.C.B., LL.D., F.R.S.
Thomas Peirson Frank.
William Thomson Halcrow.
Sir Clement Daniel Maggs Hindley, K.C.I.E., M.A.
George McCausland Hoey, B.A., B.E.
Professor Charles Edward Inglis, O.B.E., M.A., LL.D., F.R.S.
Alfred Dale Lewis, M.A.
Alexander Newlands, C.B.E.
Philip Louis Pratley, M.Eng.
Sir Leopold Halliday Savile, K.C.B.
Francis Ernest Wentworth-Sheilds, O.B.E.
Reginald Edward Stradling, C.B., M.C., Ph.D.
Sir John Edward Thornycroft, K.B.E.
William Launcelot Crosbie Trench, C.I.E., B.A., B.A.I.
Hugh Vickerman, O.B.E., D.S.O., M.Sc.
Maurice FitzGerald Wilson.

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[1] The Council commence their term of office on the first Tuesday in November, 1936.

[2] Since deceased.

Mr. F. M. G. Du-Plat-Taylor proposed and Mr. H. W. S. Husbands seconded the resolution—That the thanks of The Institution be accorded to the Scrutineers, and that the ballot-papers be destroyed. The resolution was carried by acclamation, and was acknowledged by Mr. J. D. C. Couper, who remarked that of 6,750 voting papers issued, 1,816 had been returned, of which 87 had been invalid for one reason or another, leaving 1,729 valid voting papers, or roughly 25 per cent. of those sent out. Suggestions for improving that percentage had been made, and anything that could be done to that end would be valuable.

Mr. A. W. E. Bullmore proposed and Mr. Duncan Kennedy seconded the resolution—That the thanks of The Institution be given to Messrs. P. D. Griffiths and E. W. Monkhouse, Auditors, and that they be re-appointed Auditors for the current financial year. The resolution was carried unanimously, and was acknowledged by Mr. Monkhouse.

Mr. J. S. Alford proposed "That the thanks of this Meeting be accorded to Mr. John D. Watson, President, for his conduct of the business as Chairman of the Meeting." The motion was carried by acclamation.

The Chairman thanked the members for the way in which they had received the motion. He remarked that he was not quite sure that the Statement of Accounts was as clear as it might be, and he felt that it might perhaps be simplified. He was very interested in Mr. Thorpe's account of the origin and early work of the Local Associations. He thought that the benefit which accrued to members by presenting Papers and taking an active part in the work of Local Associations should be more widely known, and gave examples of the progress that young men had made through becoming known by the Papers they had read at Meetings. He felt that the Local Associations were doing excellent work for the engineering profession.

Sir Clement Hindley proposed, and Mr. J. S. Wilson seconded, a vote of thanks to the Institution staff. At the same time, they expressed their sympathy with the Secretary in the very long and serious illness from which he had been suffering, and were very glad that he was now restored to health. The vote was carried by acclamation, and was acknowledged by Mr. E. G. Clark.

The proceedings then ended.

REPORT OF THE COUNCIL, 1935–36.

BEFORE presenting the Annual Report upon the state of The Institution, the Council wish to place on record the deep sense of loss sustained by members in all parts of the world by the death of His Majesty King George V, Patron of The Institution. The affection with which His late Majesty was regarded found full expression during the celebration of the Silver Jubilee less than a year ago.

Addresses of condolence and of loyalty, the texts of which have appeared in the Journal of The Institution,[1] were transmitted on behalf of the members of The Institution to His Majesty King Edward VIII and to Her Majesty Queen Mary.

The members will learn with satisfaction that His Majesty King Edward VIII, who, as Prince of Wales, was for many years an Honorary Member of The Institution, has been graciously pleased, on his accession to the Throne, to grant his Patronage to The Institution in succession to his father and his grandfather.

Dealing with matters which concern the administration of The Institution, the Council have to report that, following representations made to them at the last Annual General Meeting regarding the revision of the By-laws, a Special General Meeting was held on the 18th June last to consider whether members should be afforded the opportunity of recording separate votes upon each of the more important groups of modifications in the proposed new By-laws. The motion in favour of such a course having been lost, the Special General Meeting which had been summoned for the 15th October was duly held and the revised By-laws were adopted by the Corporate Members. These By-laws were approved and allowed on the 6th February by the Lords of the Privy Council, to whom they were submitted in accordance with the provisions of the Supplemental Charter of 1922, and became operative from that date.

Copies of the new By-laws and Regulations have been issued to corporate and non-corporate members.

The Council wish to take this opportunity of reviewing the changes made in respect of the Institution publications, to which reference was made in the last Report. The Journal of The Institution made its first appearance at the beginning of the Session and will henceforth be issued in November, December, January, February, March, April, June and October. It contains Papers of the kind hitherto published in the Proceedings, together with other Papers formerly published separately in pamphlet form without oral discussion, and Papers

[1] Vol. 2 (1935–36), p. 185 (March, 1936).

ordered to be printed in abstract form only. It includes a section giving accounts of the work of the Institution Research Committee as well as general information on engineering research which is being carried out in this country and abroad. Notices, hitherto issued as Sessional Notices, are also incorporated in the Journal and these have been expanded to include reports on Papers read before and visits made by Local Associations, and on other matters of current interest to the members.

One of the chief objects of the periodical publication of this Journal is to provide members with an early record of the Papers read and discussed orally at the Ordinary Meetings of The Institution, without waiting for the written discussion from overseas to be completed, such contributions being printed in subsequent numbers. The Journal also makes available to all the members the Papers formerly published separately as Selected Engineering Papers. Under present arrangements the Papers read and discussed may be divided broadly into two categories; firstly, those which deal with the records or accounts of completed engineering works or with the results of investigations or research into engineering problems, and secondly, Papers of current engineering interest likely to give rise to discussions on engineering problems which are prominently in the minds of engineers.

Mr. John D. Watson, President, delivered his Address at the Opening Meeting on the 5th November, dealing with those branches of public works which had principally engaged his attention for the past 50 years, namely, water-supply, sewerage, and sewage-disposal.

Seventeen Ordinary Meetings for the discussion of Papers have been held (the Discussion on one Paper occupying two evenings), at which the following Papers were discussed :—

SUBJECT.	AUTHOR.
"The Behaviour of Reinforced-Concrete Piles during Driving."	W. H. Glanville, D.Sc., Ph.D., M. Inst. C.E., G. Grime, M.Sc., and W. W. Davies, B.Sc. (Eng.), Assoc. M. Inst. C.E.
"Vizagapatam Harbour."	W. C. Ash, B.Sc. (Eng.), and O. B. Rattenbury, B.Sc. (Eng.), MM. Inst. C.E.
"The Energy-Output of the Coal-Miner."	Professor K. Neville Moss, O.B.E., M.Sc., M. Inst. C.E.
"Industrial, Agricultural and Domestic Heating, with Electricity as a By-Product."	S. B. Donkin, M. Inst. C.E.

"The Treatment of Mud-Runs in Bolivia."	S. W. F. Morum, B.Sc. (Eng.), Assoc. M. Inst. C.E.
"Royal Docks Approaches Improvement, London."	Duncan Kennedy, M. Inst. C.E., and H. E. Aldington, Assoc. M. Inst. C.E.
"The River Foyle Crossing (Londonderry Waterworks)."	Walter Criswell, O.B.E., M. Inst. C.E.
"A General Comparison of Gas and Electricity for Heat-Production."	A. H. Barker, B.Sc., B.A., M. Inst. C.E.
"St. Germans Sluice and Pumping-Station."	R. G. Clark, O.B.E., M. Inst. C.E.
"Effect of Flood Relief-Works on Flood-Levels below such Works."	E. C. Hillman, M.C., B.Sc., Assoc. M. Inst. C.E.
"Road Engineering Problems: Judging the Slippery Road."	R. G. C. Batson, M. Inst. C.E., G. Bird, B.Sc., and R. E. Stradling, C.B., M.C., Ph.D., D.Sc., M. Inst. C.E.
"The Construction of the Mersey Tunnel."	David Anderson, B.Sc., M. Inst. C.E.
"Some Major Problems in the Utilization of Coal."	F. S. Sinnatt, C.B., M.B.E., D.Sc.
"The Superstructure of the Island of Orleans Suspension Bridge, Quebec, Canada."	S. R. Banks, M. Eng., Assoc. M. Inst. C.E.
"Corrosion of Iron and Steel."	Sir Robert Hadfield, Bart., D.Sc., D.Met., F.R.S., M. Inst. C.E., and S. A. Main, B.Sc.
"The Rational Design of Steel Building Frames."	Professor J. F. Baker, M.A., D.Sc., Assoc. M. Inst. C.E.
"The Demolition of Waterloo Bridge."	E. J. Buckton, B.Sc. (Eng.), and H. J. Fereday, MM. Inst. C.E.

The awards for Papers read and discussed, for Papers published without oral discussion, and for Students' Papers will be announced in the October number of the Journal.

Five Informal Meetings were held during the Session; the Council learn with pleasure that many of the younger men have taken a very active interest in the discussions which have taken place. The subjects and names of the Introducers are as follows:—

"The Duties and Responsibilities of the Resident Engineer."	A. F. St. J. Kinsey, Assoc. M. Inst. C.E.

"Aerodromes."	H. A. Lewis-Dale, M.B.E., M. Inst. C.E.
"Various Advantages of Dock-Gates, Caissons and Falling Doors for Lock-Entrances and Dry Docks."	E. J. Buckton, B.Sc. (Eng.), M. Inst. C.E.
"Welding in Constructional and Repair Work."	John Miller, LL.D., B.E., M. Inst. C.E.
"Replenishment of Underground Water-Supply."	J. F. Haseldine, M.C., M. Inst. C.E.

The Forty-Second James Forrest Lecture was delivered on the 5th May, by Mr. E. F. Relf, Superintendent of the Aerodynamics Department of the National Physical Laboratory, who dealt with the subject of "Modern Developments in the Design of Aeroplanes."

The Vernon-Harcourt Lecture, on "Tidal and River Models," was delivered by Professor A. H. Gibson, M. Inst. C.E., in London, on the 18th December, and was repeated before meetings of the Local Associations at Belfast, Bristol, Birmingham, Glasgow, Leeds, Manchester and Newcastle-upon-Tyne.

A Special Lecture of much interest will be given on the 26th May, by M. Eugéne Schneider, on the subject of "Recent Developments in Metallurgy and their Influence on Engineering."

Mr. A. W. K. Billings, M. Inst. C.E., Vice-President of the São Paulo and Rio de Janeiro Tramway, Light & Power Company, Ltd., will come from South America to deliver to the members on the 16th June a lecture on "Water Power in Brazil, with Special reference to the São Paulo Development."

The Opening Meeting of the Association of London Students was held on the 20th November, when Mr. A. L. Somerville, Chairman of the Association, gave an address dealing with the historical development of roads in Great Britain. The number of Students belonging to the Association is 482. In addition to the Vernon-Harcourt Lecture, Papers were read at four Meetings, an Informal Meeting was held, and a film descriptive of the Sheffield Works of Messrs. Vickers, Ltd., was displayed. The 55th Annual Dinner of the Association was held at the Holborn Restaurant on the 20th February. Visits were paid to six places of engineering interest, and arrangements have been made for a whole-day visit on the 16th May to the Ketton Portland Cement Works, Rutland.

The Glasgow Association of Students has sustained a great loss by the death of its Chairman, Professor J. D. Cormack, C.M.G., C.B.E., M. Inst. C.E., which occurred on the 30th November. Mr. J. M. Hogg, M. Inst. C.E., became Chairman of the Association in his

place. The Association has held seven meetings, including one at Edinburgh University, and the Council are disappointed to note that no Papers were read by Students this Session. They feel that this position must be remedied in the future if the Association is to fulfil its principal object. A number of visits have also taken place, and the attendance at the 37th Annual Dinner of the Association, which was held on the 4th February, numbered 130. The number of Students supporting the Association shows a slight increase and is now 174.

The roll of the Manchester Association shows an increase of 17 over last Session and now numbers 139 Corporate Members and 108 Students. Eleven meetings have been held, at two of which Papers were read by Students, and one meeting was devoted to a discussion on the Interim Report of the Committee on Floods in relation to Reservoir Practice, the discussion being opened by Mr. W. J. E. Binnie, M. Inst. C.E., Chairman of the Committee. Three visits to works have been made. The Annual Dinner was held on the 5th February, when the attendance was 112.

The membership of the Birmingham and District Association again shows an appreciable increase over last year, the membership, including all grades, numbering 318, being 193 Corporate Members and 125 Students, as compared with 297 in the Session 1934–35. Twelve meetings have been held, including a Joint Students' Meeting with the Students of the local branches of the Institutions of Electrical and Mechanical Engineers, and a meeting of Students of the Association which took the form of a supper followed by a discussion on "Alterations to Existing Means of Internal Communications to suit Modern Methods of Transport." Five visits to works took place, and on the 1st March the Chairman and other members of the Association attended a service at Handsworth Parish Church arranged to commemorate the Bicentenary of the birth of James Watt. The Annual Dinner was held on the 12th December, when 139 members and guests were present.

The Newcastle-upon-Tyne and District Association has held fifteen meetings, including three Students' Meetings, and six meetings have also been held at Stockton-on-Tees, the second centre of the Association. The roll of the Association consists of 74 Corporate Members and 78 Students. Owing to the death of His Majesty King George, the Annual Dinner of the Association, which had been arranged for the 20th January, was abandoned.

The Yorkshire Association numbers 201 Corporate Members and 81 Students. Nine meetings have been held, including a joint meeting with the local branches of the Institution of Structural Engineers and the Institution of Municipal and County Engineers. The Annual Dinner was held on the 2nd April at Leeds.

The Bristol and District Association has 80 Corporate Members and 72 Students on its roll, compared with 73 Corporate Members and 67 Students last Session. Six meetings have been held, including one at Gloucester with very good results, and the attendance at the meetings has shown an improvement. The Annual Dinner, held on the 16th December, was again a successful function and was attended by 88 members and guests.

The South Wales and Monmouthshire Association, which numbers 139 Corporate Members and 48 Students on its roll, has held six meetings during the Session, four at Cardiff and two at Swansea, and the attendance at these meetings has been satisfactory. The Association's Prize of £5 has been divided between Mr. R. H. Edwards, Assoc. M. Inst. C.E., for his Paper on "The Organization and Work of the Divisional Docks Engineer's Office, Eastern Ports, Great Western Railway" and Mr. J. McEwen King, Assoc. M. Inst. C.E., for his Paper on "Recent Highway Schemes in Newport."

The Northern Ireland Association has a total membership of 98, composed of 80 Corporate Members and 18 Students, as compared with 88 Corporate Members and 18 Students last year. Ten meetings have been held and the Annual General Meeting will take place on the 20th May. One visit to works was paid last summer, and the second Annual Dinner of the Association was held on the 8th November and was attended by 67 members and guests.

The membership of the Buenos Aires Association at the end of 1935 numbered 86, being 73 Corporate Members and 13 Students, as compared with 88 at the end of the previous year. Three meetings were held during the Session, and three visits to works were made, including a four-day visit to various works of interest at Tucuman. The Association had the pleasure of welcoming and entertaining Sir Richard Redmayne, Past-President, to luncheon during his visit to South America last year.

The Session of the Malayan Association closed in September, 1935, the total membership of the Association at that date being 106, consisting entirely of Corporate Members. A number of meetings and visits to works in conjunction with the Engineering Association of Malaya have taken place and the premium for the best Paper received during 1934 has been awarded to Mr. N. H. Taylor, Assoc. M. Inst. C.E., for his Paper on "The Alleviation of Flooding in Singapore."

The members of the Shanghai Association have, under a scheme of co-operation which has been approved by the Council, taken part during the past Session in a number of joint meetings and visits to works with the Engineering Society of China and the Local Centre of the Institution of Electrical Engineers. At one of the meetings

a Paper on "Dredging of the Yangtze Bar" was read by Dr. H. Chatley, M. Inst. C.E., and it is hoped that further Papers from members of the Shanghai Association will be forthcoming during the course of next Session.

The Research Committee, as reconstituted towards the end of last Session under the chairmanship of Sir Clement Hindley, M. Inst. C.E., has examined a number of proposals for the investigation of specific research problems in regard to which further information was considered likely to prove of practical value to engineers. In the result, Sub-Committees composed of technical experts have been appointed to investigate and report on the following problems :—

1. The measurement of wave pressures on sea structures.
2. The effect of vibration methods of deposition on concrete.
3. The effect of soils containing sulphate salts on concrete and metal pipes.
4. The use of self-contained breathing apparatus in sewers, tunnels and the like.
5. The design and construction of reinforced-concrete structures for storing liquids.
6. The possible injury to metal water-pipes and mains through the earthing thereto of electrical installations.
7. The design of efficient forms of fish passes in connection with river-development works.
8. Velocity formulas for open channels and pipes.

In addition to the above, the Research Committee, after negotiation with the bodies concerned, has assumed responsibility for the direction of the following researches :—

9. Investigations into the behaviour of reinforced-concrete piles during driving, instituted by the Federation of Civil Engineering Contractors in conjunction with the Building Research Board.
10. Investigations into the special properties required in cement for large dams, instituted by the British Committee on Large Dams of the World Power Conference.
11. Investigations on earth pressures, which were being carried out on the initiative of the British Association for the Advancement of Science.

In each of the above three cases the Sub-Committees continued their work with the same personnel as the existing committees.

The general policy laid down for the effective prosecution of the Research Committee's work is to invite the co-operation of other

Institutions and bodies interested in any particular research by the appointment of representatives on the Sub-Committee carrying out the investigation, and, where expenditure of money for experiments, etc., is involved, to seek their financial assistance. The Council has decided to allot a sum of £1,000 for the first year and £2,000 per annum for the next 4 years for the promotion of research work vouched for by the Research Committee and approved by the Council. A special assistant has been appointed as a member of the Institution Staff to devote his whole time to the secretarial work of the Research Committee and its various Sub-Committees.

The Research Committee have also dealt with other matters referred to them by the Council and have in particular examined a number of British Standards Institution Specifications and made recommendations in connection therewith.

The 15th Report of the Committee on the Deterioration of Structures Exposed to Sea Action, which summarized the results of the various investigations obtained up to 1935, has been published.

Periodical reports have been received giving results of examination of variously-treated timber specimens exposed at Auckland, Wellington, Colombo and Takoradi, and dealing with the examination of incised and creosoted British Columbia fir specimens after 11 months' exposure at Colombo. Detailed reports have been received from Professor George Barger, F.R.S., on his examination of specimens received from Leith and from Takoradi, and on the work carried out by him since 1933 on the protection of timber from marine borers.

Reports have been received describing the results of the examination of iron and steel specimens at Auckland, Plymouth and Colombo.

There have also been received periodical reports on reinforced concrete specimens and piles exposed at Sekondi, Gold Coast, and from the Building Research Station on the inspection of concrete piles exposed at Sheerness and Watford.

On the 30th April, 1935, a Committee was appointed to draw up, for the guidance of engineers and contractors, regulations for work carried out under compressed air. The Committee, which consisted of representatives of The Institution, of members nominated by the Federation of Civil Engineering Contractors, and of other persons co-opted as experts in problems arising out of the physiological effects of compressed air, has presented its report. In adopting the report, the Council wish to record their appreciation of the valuable services of the Committee and to express the thanks of The Institution to Mr. David Anderson, M. Inst. C.E. (the Chairman), and to the other members.

The Council also wish to express the deep regret with which they

learned of the death of Professor J. S. Haldane, C.H., F.R.S., a member of the Committee, which occurred after the publication of the report. Professor Haldane, with his unique experience of physiological problems, placed his knowledge unreservedly at the disposal of the Committee and was of the greatest assistance when the question of adapting the method of stage decompression as used in deep-sea diving to engineering work was under consideration.

The Report is published and is being sold under the authority of the Council by Messrs. William Clowes and Sons.

Early in 1935, a Joint Select Committee was set up by the Government to receive evidence on Water Resources and Supplies in England and Wales. Representations were made to the Joint Committee that The Institution and the Association of Consulting Engineers should be given an opportunity of submitting evidence, and, as a result, representatives of the two bodies were heard by the Joint Committee. A memorandum subsequently prepared by the Ministry of Health in accordance with a resolution passed by the Joint Committee was circulated, and the representatives of The Institution and of the Association of Consulting Engineers submitted comments thereon and are expecting shortly to give further evidence before the Joint Committee, which has given sympathetic consideration to their criticisms.

In September, 1935, The Institution was invited by the Ordnance Survey Departmental Committee of the Ministry of Agriculture and Fisheries to submit evidence upon the measures deemed necessary to accelerate the revision of Ordnance Survey maps and other matters connected with their publication. A special Committee of the Council was appointed to deal with this matter and a statement of their comments on the subject was prepared and placed before the Departmental Committee. Subsequently two members of the Special Committee appeared before that Committee and supplemented these comments by oral evidence.

Owing to the national mourning, the Annual Dinner which had been fixed for the 4th March was postponed to the 29th April. The Dinner was held at the Savoy Hotel and was attended by 414 members and guests.

A Conversazione took place at the Institution on the 12th June, 1935, and was attended by 2,517 members and guests.

Various nominations and appointments have been made or renewed by the Council during the past year, and The Institution is or has been represented on advisory or administrative bodies and committees by the following members :—

Royal Commission for Exhibition of 1851	The President.
Grant Committee of the Royal Society	The President.

General Board of National Physical Laboratory	Sir Clement D. M. Hindley, K.C.I.E. Sir Richard A. S. Redmayne, K.C.B.
Admiralty Selection Board for the Appointment of Assistant Civil Engineers	Sir Alexander Gibb, G.B.E., C.B., F.R.S. Sir Cyril R. S. Kirkpatrick.

Committees of the Department of Scientific and Industrial Research :—

Structural Steel Research Committee	David Anderson. Ralph Freeman. B. L. Hurst.
Committee on Testing Work for the Building Industry	The President or his Deputy.
Royal Engineer Board	Sir Brodie H. Henderson, K.C.M.G., C.B.
Mechanisation Board, Army Council	W. G. Wilson, C.M.G.
Advisory Panel on Transport (Ministry of Transport)	O. R. H. Bury. Col. R. E. B. Crompton, C.B., F.R.S. Sir John P. Griffith. J. A. Saner.
Ministry of Health Technical Committee on Materials for the economic construction of Flats	W. L. Scott.
Home Office Sub-Committee on Air Raid Precautions	Sir Leopold H. Savile, K.C.B.
Science Museum Advisory Council, Board of Education	Professor C. E. Inglis, O.B.E., F.R.S.
Court of the University of Bristol	Raymond Carpmael.
Court of the University of Liverpool	Thomas Molyneux, O.B.E.
Court of the University of Sheffield	Sir William H. Ellis, G.B.E.
Governing Body of the Imperial College of Science and Technology	Sir George W. Humphreys, K.B.E.
Council of the City and Guilds of London Institute	The President.
City and Guilds of London Institute Fellowship Selection Committee	Sir Brodie H. Henderson.
Highway Engineering Advisory Committee (University of London)	Sir Brodie H. Henderson.

Court of University College, Southampton	F. E. Wentworth-Sheilds, O.B.E. W. J. Taylor, O.B.E.
Governing Body of the School of Metalliferous Mining, Cornwall	J. G. Lawn, C.B.E.
Thomason College, Roorkee, Advisory Council	G. McC. Hoey.
Old Centralians Committee on Memorial to Dr. W. C. Unwin	Sir Richard A. S. Redmayne. Sir Charles L. Morgan, C.B.E. The Secretary.
Engineering Advisory Committee, Huddersfield Engineering College	V. Turner. J. Urquhart.
Technical Advisory Committee of the Institute for Research in Agricultural Engineering at the University of Oxford	Sir John E. Thornycroft, K.B.E.
Engineering Joint Council	Sir George W. Humphreys. Sir Leopold H. Savile.
Parliamentary Science Committee	Sir George W. Humphreys. Sir Murdoch MacDonald, K.C.M.G., C.B., M.P.
Council of the London Society	T. H. Bailey.
Governing Body of the Denning Trust	A. E. Cornewall-Walker.
Tribunal of Appeal, London Building Act, 1930	Sir George W. Humphreys.
Royal Institute of British Architects Sub-Committee on Information Bureau on Building Materials and their Uses	R. E. Stradling, C.B., M.C.
Conference on Proposals for the Study of Materials and Construction and their Testing	R. E. Stradling.
Alloys and Iron Research Committee of the Institution of Mechanical Engineers	Sir Robert A. Hadfield, Bt., F.R.S.
British Cast Iron Research Association	Sir Robert A. Hadfield.
Permanent Commission of International Navigation Congresses	Sir Cyril R. S. Kirkpatrick. N. G. Gedye, O.B.E.
World Power Conference British National Committee	S. B. Donkin.
World Power Conference Sub-Committee on Special Cements	W. T. Halcrow.

International Electrotechnical Commission on Steam Turbines	I. V. Robinson.
International Electrotechnical Commission Advisory Committee on Internal-Combustion Engines	W. A. Tookey.

The Institution is represented as follows on Councils of the British Standards Institution :—

General Council	Sir Cyril R. S. Kirkpatrick.
Engineering Divisional Council	S. B. Donkin. R. G. Hetherington, C.B., O.B.E. A. Newlands, C.B.E.

and has also representatives on numerous Committees, Sub-Committees and Panels.

Main Committee of the Canadian Engineering Standards Association	H. H. Vaughan.

The Council nominated Sir John P. Griffith and Mr. E. F. C. Trench, Past-Presidents, and Professor John Purser, M. Inst. C.E., to represent The Institution at the Centenary Celebration of The Institution of Civil Engineers of Ireland, held in Dublin in August, 1935 ; and Professor Henry Louis at the 75th Anniversary of the foundation of the Svenska Teknologföreningen (the Swedish Society of Engineers and Architects), to be held in Stockholm in May, 1936.

In response to an invitation from the Chairman of the British Committee of the Second International Association for Testing Materials, the Council have appointed Mr. R. H. H. Stanger, Assoc. M. Inst. C.E., to be a representative on the Organizing and Reception Committee of the Congress to be held in London in 1937.

The bicentenary of the birth of James Watt, which fell on Sunday, the 19th January, was commemorated by the placing of a wreath on the statue of Watt in the Chapel of St. Paul, Westminster Abbey, at the conclusion of evensong, by Mr. Watson, the President, who was accompanied by the Presidents or representatives of a number of other engineering societies.

Reference was made in last year's Report to the action which had been taken to place the conditions governing the award of the C. C. Lindsay Civil Engineering Scholarships on a broader basis. The Council are pleased to report that suitable applicants for these scholarships are now coming forward, and during the past year a scholarship of £35 per annum for two years has been awarded to Mr. Thomas Alexander Kerr, Stud. Inst. C.E., and scholarships of

£25 per annum for two and four years respectively have been awarded to Mr. James Melville Niven and Mr. Gavin Harvie, Studs. Inst. C.E.

The William Lindley Scholarships have been vacant since 1935 and the Council regret that more candidates, who must be the children of corporate members, are not forthcoming to take advantage of these valuable aids to securing scientific education and practical engineering training.

The Council have received the report of the judges on the entries received for this year's Charles Hawksley Prize Competition and regret to find that, although up to the average in point of numbers, the designs did not reach the standard shown during the past few years. No entry was found to be of sufficient merit to warrant the award of the Charles Hawksley Prize, but, on the judges' recommendation, two candidates have been honourably mentioned and have been granted £50 each in recognition of their careful work.

During the year 657 volumes were presented to the Library and 25[illegible] were purchased, making a total, on the 31st March, 1936, of 61,383.

The number of applications received from members for books from the Loan Library total 1,732 for the year. The Loan Library Catalogue was issued during the year and 487 applications were received for copies.

Gifts received by The Institution during the year include a photograph of the late Sir Charles Hutton Gregory, K.C.M.G. (Past-President), presented by Mr. Charles Froom; a bust of J. R. McClean (Past-President), presented by Mr. W. N. McClean (Assoc. M. Inst. C.E.); and a bust of John Rennie and 34 volumes of reports and [illegible] letter books, some of which originally belonged to Rennie, presented by Mr. John A. Rennie.

The lease of No. 1 Great George Street having expired, the premises are being demolished, and the N.W. corner of The Institution building will now be completed. A considerable portion of the ground rendered available will, in accordance with the agreement entered into when The Institution acquired its present site, be handed over to the Westminster City Council for street widening.

A cinematograph projector for use with standard films has been installed and a licence obtained from the London County Council. The Council hope that the members will take full advantage of the opportunity thus afforded to show moving pictures as well as still pictures to illustrate the Papers read and discussed.

Finance.

The following is a statement of receipts and payments for the year ended 31 March, 1936, the corresponding amounts for the financial year 1934–1935 being given in brackets. The complete accounts, which have been duly audited, are given in the Appendix.

The Receipts were:—On General Income Account £43,915 12*s*. 7*d*. (*£42,754 18s. 1d.*) of which £38,031 2*s*. 4*d*. (*£36,306 7s. 5d.*) represents subscriptions and fees apportioned to the Financial Year, £2,133 16*s*. 9*d*. (*£2,119 5s. 6d.*) Dividends and Interest on Investments, £203 0*s*. 9*d*. Income Tax on Investments refunded for the year 1934–1935, £207 12*s*. 3*d*. (*£162 8s. 2d.*) rent of No. 1 Great George Street.

The Expenditure was:—On General Account £42,618 13*s*. 2*d*. (*£45,236 18s. 3d.*), which included provision for Publications £10,000 0*s*. 0*d*. (*£14,888 14s. 1d.*) and Interest on Loan under amortization £234 6*s*. 6*d*. (*£305 3s. 1d.*).

The sum of £1,669 1*s*. 7*d*. (*£1,601 0s. 9d.*) was applied to the extinction of the Loan.

On Trust Funds Income Account there was received a total of £1,224 7*s*. 10*d*. (*£1,217 17s. 3d.*), and the Expenditure amounted to £977 8*s*. 2*d*. (*£1,092 1s. 6d.*).

Contributions amounting to £630 4*s*. 2*d*. (*£693 12s. 2d.*) were received from Home and Overseas Harbour and Dock Authorities towards the cost of the research into the Deterioration of Structures exposed to Sea Action. The expenditure during the year was £485 13*s*. 2*d*. (*£671 14s. 2d.*).

During the Session which ended on the 30th April, 1936, the Council have considered 586 proposals for election (including cases postponed from previous years). Ninety-five recommendations for the transfer of Associate Members to the class of Members were also considered.

For the year which ended on the 31st March, 1936, the election comprised 1 Honorary Member, 11 Members, 399 Associate Members, and 4 Associates; 387 candidates were admitted as Students, and the names of 1 Member, 15 Associate Members, and 1 Student were restored to the Roll. From this addition of 819 must be deducted the deaths, resignations, and erasures, amounting to 372 in all, showing a net addition of 447. The Council also transferred 98 Associate Members to the class of full Members.

The number of candidates presenting themselves for the October, 1935, Examination was 519, namely, 87 for the Preliminary Examination and 432 for the Associate Membership Examination. The entries for the April, 1936, Examinations were 193 for the Preliminary Examination and 508 for the Associate Membership Examination.

Bayliss Prizes of the value of £15 have been awarded to Mr. James William Milne and Mr. John Woollhead Hooper, B.Sc., Studs. Inst. C.E., in respect of Sections A and B of the Associate Membership Examination for April and October, 1935, respectively, and Mr. Lyndsay Norman Prismall has been honourably mentioned for his performance in the latter examination.

The Council wish to draw attention to the modifications which have been made in the revised By-laws in connection with the conditions for election into The Institution. Election as full Members of persons who can show that they have held for 15 years positions of full responsibility in the design or execution of important engineering work has now been restricted to persons who are not less than 50 years of age. Exception is made, however, in the case of those who are eminent in the profession and who "have made some noteworthy contribution to the science of engineering or materially advanced the practice of engineering from the technical point of view." For the purpose of this By-law, professors of engineering at recognized colleges and persons engaged on important engineering research are regarded as employed "in the design of important engineering work."

Likewise, in connection with election to Associate Membership, teachers of engineering at recognized colleges and engineering research workers are regarded as employed "in the design of engineering work," and post-graduate engineering study or engineering research may be accepted as fulfilling part of a candidate's engineering experience.

The Council are desirous that these facts should become fully known, as they are aware that there are many persons possessing such qualifications which have hitherto been unacceptable under the By-laws. Now that the basis of qualifications has been extended in this direction, the Council hope that all such persons will seek election into The Institution.

The Council have decided that in principle there is nothing inconsistent with the By-laws in the election to corporate membership of The Institution of engineers holding permanent commissions in H.M. Forces, provided that their qualifications in education and practical training comply in every respect with the requirements of the By-laws.

Enumeration. The Roll of The Institution on the 31st March, 1936, stood at 11,354, the changes which took place in it during the year ended on that date being shown in the Table on the opposite page.

The Roll at this date is 11,357.

The Council record with especial regret the deaths of Sir Richard Tetley Glazebrook, K.C.B., K.C.V.O., M.A., D.Sc., F.R.S.; Admiral of the Fleet The Rt. Hon. Earl Jellicoe of Scapa, O.M., G.C.B., G.C.V.O., Honorary Members; William Ferguson, M.A., B.A.I., Ernest Prescot Hill, and Sir Hugh Reid, Bart., C.B.E., LL.D., former Members of Council.

	1 April, 1934, to 31 March, 1935.						1 April, 1935, to 31 March, 1936.					
	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.
umbers at commencement	18	2217	6835	62	1733	10,865	16	2222	6937	56	1676	10,907
ransferred to Members	..	69	69	..	..		..	98	98	..	..	
lections	..	18	317	..	..	648	1	11	399	4	..	819
dmissions	..	..	..	..	295		..	..	..	..	387	
estored to Roll	..	2	14	..	2		..	1	15	..	1	
eceased	2	66	53	2	2	606	2	53	57	2	4	372
esigned	..	13	64	3	24		..	13	47	..	19	
rased	..	5	41	1	11		1	5	27	..	11	
lected as Associate Members	..	..	..	..	86		..	..	..	..	122	
emoved—over age	..	..	..	..	227		..	..	..	..	3	
ailed to complete	..	..	2	..	..		..	..	1	..	1	
ailed to comply (Studentship)	..	..	..	..	4	+ 42	..	..	..	..	4	+447
umbers at termination	16	2222	6937	56	1676	10,907	14	2261	7121	58	1900	11,354

The full list of deaths is as follows (*E. refers to election, T. to transfer,* Deaths.
nd A. to admission) :—

Honorary Members (2).—*Sir* Richard Tetley Glazebrook, K.C.B., K.C.V.O., I.A., D.Sc., F.R.S. (E. Assoc. 1904. Hon. M. 1923); *Admiral of the Fleet he Rt. Hon. Earl* Jellicoe of Scapa, O.M., G.C.B., G.C.V.O. (E. 1919).

Members (53).—Henry Adams (E. 1871. T. 1883); John Alexander (E. 1905. . 1922); David Balfour (E. 1892. T. 1901); Edward Cecil Bartlett (E. 1895. . 1909); William Frederick Beardshaw (E. 1908); Francisco de Paula Bicalho E. 1910); Frederick Thomas Bowler, B.Sc. (E. 1908. T. 1933); Charles imond Horatio Braine (E. 1903. T. 1928); George James Cotton Broom E. 1877. T. 1892); Arthur Brown (E. 1881. T. 1885); William Carnegie, I.B.E. (E. 1902. T. 1919); Edward Richard Carolin (E. 1883. T. 1888); Villiam Hastings Cavendish, B.Sc. (E. 1902); Leonard Cooper (E. 1894. T. 913); *Professor* John Dewar Cormack, C.M.G., C.B.E., D.Sc. (E. 1902. T. 912); Thomas Hodges Deakin (E. 1891. T. 1907); Clement Dunscombe, M.A. E. 1879. T. 1883); Walter Emmott (E. 1893. T. 1905); Thomas Milnes avell (E. 1875. T. 1894); William Ferguson, M.A., B.A.I. (*former Member of ouncil*) (E. 1880. T. 1893); Gerald FitzGibbon (E. 1883. T. 1898); Thomas hillip Francis (E. 1924. T. 1932); Edward Gabbett (E. 1900. T. 1906); dward Thomas Mervyn Garlick (E. 1919. T. 1927); Percy Carlyle Gilchrist, .R.S. (E. 1884); *Professor* John Goodman (E. 1887. T. 1900); Thomas oulden (E. 1892. T. 1914); John Graham (E. 1891. T. 1897); Malcolm rant-Dalton (E. 1881. T. 1890); Sri Krishna Gurtu (E. 1919); Ernest Prescot

Hill (*former Member of Council*) (*E.* 1891. *T.* 1894); Edward Arthur Hoa (*E.* 1879. *T.* 1895); John Henry Holmes (*E.* 1884. *T.* 1897); Harry Robe Kempe (*E.* 1878. *T.* 1904); George Bertram de Betham Kershaw (*E.* 1914 Donald Alexander Matheson, M.V.O. (*E.* 1887. *T.* 1895); Henry Miller (*E.* 18 *T.* 1886); Adrien Charles Mountain (*E.* 1885); George Moyle (*E.* 1880. *T.* 189 William Thomas Olive (*E.* 1879. *T.* 1891); *Sir* Hugh Reid, *Bart.*, C.B.I LL.D. (*former Member of Council*) (*E.* 1897); George Remington (*E.* 1920 Edwin Wilbur Rice, *Jun.* (*E.* 1897); Arthur Burden Campbell Rogers, C.B. (*E.* 1899); Alfred Saxon (*E.* 1924); Domingos Sergio de Saboia e Silva (*E.* 18 *T.* 1888); Alexander Irving Sleigh, F.C.H. (*E.* 1907. *T.* 1916); William Charl Ernest Smith (*E.* 1886. *T.* 1910); Albion Thomas Snell (*E.* 1890. *T.* 1905 Johann Philip Edmond Charles Stromeyer, O.B.E. (*E.* 1885. *T.* 1897); Geor William Sutcliffe (*E.* 1877. *T.* 1893); Henry Ward (*E.* 1883. *T.* 1894); Oliv Ernest Winter (*E.* 1894. *T.* 1913).

Associate Members (57).—Frank Beck (*E.* 1915); Newton Bennaton (*E.* 1882 Philip James Bevan (*E.* 1897); Ernest Spencer Bourne (*E.* 1907); Willia Arthur Chettle (*E.* 1900); William Edward Corrie, M.B.E., B.Sc. (*E.* 1902 William Cubitt (*E.* 1881); Stephen William Dassenaike (*E.* 1930); Willia Henry Duckworth (*E.* 1915); Alfred John Duncan (*E.* 1890); Clement Bea mont Ferry (*E.* 1902); Ernest Gordon Fraser (*E.* 1881); Hugh Lawrence Franc B.Sc. (*E.* 1921); Raymond Gill (*E.* 1918); Percy Murly Gotto (*E.* 1884 Frederick Morris Greenhill (*E.* 1885); Henry Lipson Hancock (*E.* 1895); Arthu Harley (*E.* 1888); Charles Francis Heathcote (*E.* 1893); *Professor* Dani Henninger (*E.* 1893); Henry Joseph Higgs, O.B.E., A.M., B.A. (*E.* 1917 Frank Ernest Hobley (*E.* 1924); Ellice Martin Horsburgh, M.A., D.Sc. (*E.* 1900 Frank Geere Howard (*E.* 1886); George Jameson (*E.* 1880); Albert Wilto King, B.C.E. (*E.* 1914); George John Kingsnorth (*E.* 1919); Arthur Robe Lungley (*E.* 1883); Frederic Thomas Maltby (*E.* 1904); George Barker Merce (*E.* 1883); Ernest Sherwood Woollard Moore (*E.* 1894); John Claridge Nicho (*E.* 1896); Horace Watson Nicholson, C.I.E., B.Sc. (*E.* 1910); George To Ogilvie (*E.* 1886); Richard Oliver Ormerod (*E.* 1885); William Osmond (1887); Henry Villiers Pegg (*E.* 1890); Samuel de Perrot (*E.* 1889); Arnol Henry Perry (*E.* 1889); Edward Cooper Poole (*E.* 1892); Geoffrey Porter (1902); John Marshall Rodger, B.Sc. (*E.* 1914); Robert Rowand (*E.* 1900) Walter Saise (*E.* 1916); Henry Rodolph de Salis (*E.* 1893); James Thoma Scarlett, B.Sc. (*E.* 1918); Jesse French Scott (*E.* 1886); Herbert Severn (1915); James Tilley Shand (*E.* 1882); James Brown Stephen (*E.* 1883); S Henry Cracroft Trollope, *Bart.* (*E.* 1904); Leslie Ralph Walklin, B.Sc. (*E.* 1930) Kaichi Watanabe, B.Sc. (*E.* 1888); Augustus While (*E.* 1909); Ernest Charl Winter (*E.* 1907); Ronald William Wynn, M.A. (*E.* 1934); Fred Spencer Yat (*E.* 1887).

Associates (2).—Gustav Behrens (*E.* 1890); Alfred Leonard Lawley (*E.* 1907)

Students (4).—Roger Gordon Jones, B.Sc. (*A.* 1932); Griffith David Llewelly B.Sc. (*A.* 1931); Eric William Anthon Nobili (*A.* 1932); Edwin Mather Thom son (*A.* 1931).

The following resignations have been received :—

Resignations.

Members (13).—Charles Henry James Clayton, M.B.E. (*E.* 1921); Richa Mountford Deeley (*E.* 1906); James Espinasse, B.A., B.A.I. (*E.* 1906. *T.* 1922 John Willim Hall (*E.* 1899. *T.* 1922); John Angus Hay (*E.* 1897. *T.* 1913 Hugh Torrance Ker (*E.* 1890. *T.* 1902); *Major* Robert Ernest Tomlin Mone Shewan, R.E. (*ret.*) (*E.* 1925); Joseph John Mullaly, C.I.E. (*E.* 1892); Robe Lloyd Roberts (*E.* 1911); Francis Blewett Shaw (*E.* 1908. *T.* 1919); Herbe

Walter Stride (*E.* 1894. *T.* 1908); Arthur Surveyer, D. Eng. (*E.* 1919); Edward Hume Townshend, B.A.I. (*E.* 1900. *T.* 1923).

Associate Members (47).—Eric Pearson Adair (*E.* 1914); Gerald Arthur Adair, M.A.I. (*E.* 1914); Robert Walton Ball (*E.* 1924); Philip Marfleet Battle (*E.* 1908); Albert Matthew Brookes (*E.* 1904); Harry Vernon Butterfield (*E.* 1917); Thomas Copley Calvert (*E.* 1894); Herbert George Clarke (*E.* 1896); William Alfred Henry Clarry (*E.* 1898); William Davidson (*E.* 1913); Charles Herbert Dobbs (*E.* 1909); Beverley Carthew Covell (*E.* 1908); Tudor Garfield Cule, B.Sc. (*E.* 1907); Andrew Duncan (*E.* 1891); Albert Edward Gardener (*E.* 1907); Andrew Gray (*E.* 1898); Andrew Paton Gray, *Jun.* (*E.* 1921); Richard Guest (*E.* 1894); Alfred Woods Hanckel, M.Sc. (*E.* 1900); Albert Francis Louis Helfrich Harrison (*E.* 1909); Owen Walter Henman (*E.* 1909); Arthur Cecil Hewitt (*E.* 1914); John Percy Maghull Hibbert, M.C., M.A. (*E.* 1911); William George Hibbins (*E.* 1900); Harry Hunter, B.Sc. (*E.* 1919); Braham Taylor Judah (*E.* 1908); Edward Knapman (*E.* 1904); Hugh Leader (*E.* 1900); Geraint Wynne Madoc-Jones, B.Sc. (*E.* 1929); Teja Singh Malik, C.I.E., B.Sc. (*E.* 1917); Francis Joseph Mills (*E.* 1911); Arthur William Nye (*E.* 1897); Alfred William Barr Pallister (*E.* 1923); Arthur Palmer, B.A.I. (*E.* 1907); John Phillips (*E.* (1894); Reginald Braham Robinson (*E.* 1908); Ernest Gravenor Rodwell, F.C.H. (*E.* 1907); Joseph Rushworth (*E.* 1895); William Valentine Shearer, B.Sc. (*E.* 1908); Robert Beevor Simmers (*E.* 1906); George Eric Rowland Slade, B.Sc. (*E.* 1912); Alfred Leonard Stocken (*E.* 1892); William Edmond Clason Thomas (*since reinstated*) (*E.* 1887); Clarence Edward Thompson (*E.* 1907); Arthur Trevor-Roberts, B.Sc. (*E.* 1907); Eric Franklin Winser, B.A. (*E.* 1910); William Henry Young, B.C.E. (*E.* 1931).

Students (19).—Noel Jack Arney (*A.* 1932); Erich Carl Albert Backhaus (*A.* 1934); William Stuart Bennett (*A.* 1931); Pierre Charles Bisson (*A.* 1932); Terence David Brian Blake (*A.* 1933); Charles Ryle Gibbs (*A.* 1933); Louis Bernard Hall (*A.* 1930); Rex Hammond (*A.* 1931); Robert Rossiter Holder (*A.* 1932); Alban Alexander le Roux (*A.* 1933); Anthony Featherstonehaugh Martindale, B.A. (*A.* 1932); Ronald William Plowright (*A.* 1931); Kenneth Walter James Rice (*A.* 1931); Frederick Denney Scott (*A.* 1927); David Hawkins Smith (*A.* 1928); Mansel Thomas, B.Sc. (*A.* 1933); Arthur Clayton Tingley (*A.* 1935); John Gardner Walter (*A.* 1931); Edward Denis Ward (*A.* 1931).

[APPENDIX.

APPEN

BALANCE SHEET

	£ s. d.	£ s. d.
To Institution Capital Account and Building Fund, *as detailed on pages 590 and 591* .	..	413,103 13 10
„ Loan on Security of Institution Buildings—		
As per last account	7,114 2 7	
Less repaid during year	1,669 1 7	
		5,445 1 0
„ Creditors	..	171 9 7
„ Repairs and Renewals Reserve, *as detailed on pages 590 and 591*	..	6,028 17 7
„ Research Reserve—*as detailed on pages 596 and 597*	..	1,200 0 0
„ W. A. P. Tait Legacy, including unspent Income	..	630 9 2
„ Revenue, etc., Items in Suspense . . .	..	83 11 10
„ Sea Action Committee Account—*as detailed on pages 596 and 597*	..	2,121 18 3
„ Trust Funds, Capital and Income Accounts—		
Capital Accounts—*as detailed on pages 594 and 595, invested per contra*	36,449 17 7	
Income Accounts—Balances unexpended—*as detailed on pages 596 and 597* . .	1,939 3 9	
		38,389 1 4
„ Institution Revenue in Suspense—		
Proportion of 1936 Subscriptions applicable to the nine months from 1st April to 31st December, 1936	15,136 17 0	
Subscriptions received in advance . . .	94 5 6	
		15,231 2 6
„ General and Contingency Reserve (being accumulations of Income over Expenditure from 1st April, 1935)		
For Year to 31st March, 1936 . .	..	1,296 19 5
		£483,702 4 6

AUDITOR

We have audited the above Balance Sheet dated 31st March, 1936, and have obtain
is properly drawn up so as to exhibit a true and correct view of the state of The Institutio
shown by the books of The Institution.

London, 7th May, 1936.

DIX.

31st MARCH, 1936.

				£	s.	d.
By Expenditure on Institution Buildings, including Cost of Site, *as per last account*				352,072	7	2
,, Institution Investments (including those held in respect of Repairs and Renewals Reserve) at cost, *as detailed on page 598*				70,808	14	8
Note.—*The value of these Investments at ruling prices on 31st March, 1936, amounted approximately to £73,007.*						
,, W. A. P. Tait Legacy Investment at cost, *per page 598*				514	11	8
,, Trust Funds Investments, etc.—						
Capital :—						
Investments, *as detailed on pages 594 and 595*			£36,449 17 7			
Unexpended Income :—						
Investments, *as detailed on page 595*		£214 8 2				
Cash at Bank—						
On Deposit a/c	1,650 0 0					
,, Current a/c	74 15 7					
		1,724 15 7				
			1,939 3 9			
				38,389	1	4
,, Debtors				897	6	8
Cash at Bankers, and in Hand—						
At Bank—Deposit and Current Accounts :—						
Institution Funds			16,478 17 0			
Sea Action Committee Account			2,121 18 3			
			18,600 15 3			
In Hand			4 15 6			
				18,605	10	9
Publications Account—						
Balance overspent, *per page 590*				2,414	12	3
Note.—*No value has been attached, for the purpose of this Balance Sheet, to the Books, Furniture, Pictures, Sculpture, etc., belonging to The Institution.*						
				£483,702	4	6

H. H. Jeffcott, *Secretary.*

PORT.

he information and explanations we have required. In our opinion such Balance Sheet
irs according to the best of our information and the explanations given to us, and as

Percival D. Griffiths, F.C.A.
E. W. Monkhouse, M. Inst. C.E. } Auditors.

INSTITUTION CAPITAL ACCOUNT

	£	s.	d.
To Balance *carried down*	413,103	13	10
	£413,103	13	10

RESERVE FOR REPAIRS AND RENEWALS TO

	£	s.	d.
To Expenditure during the Year	2,112	10	7
„ Balance *carried down*	6,028	17	7
	£8,141	8	2

PUBLICATIONS

	£	s.	d.
To Expenditure on Publications for the Year—			
Journal	4,108	9	1
Minutes of Proceedings	4,542	12	11
Selected Engineering Papers	827	19	4
Sessional Notices	505	8	1
Charters, By-laws and Lists of Members	456	7	9
Engineering Abstracts	1,664	14	1
Salaries, Clerical Pay and Pensions Premiums	2,519	0	9
	£14,624	12	0
Less Credits for Advertisements, Sales, Contributions, etc.	2,209	19	9
	£12,414	12	3
To Balance *brought down—as per Balance Sheet, page 589*	£2,414	12	3

AND BUILDING FUND, 31st MARCH, 1936.

	£	s.	d.
By Balance—*per last account*	413,088	13	7
,, Portion of Legacy under the Will of Sir Dugald Clerk, M. Inst. C.E.	15	0	3
	£413,103	13	10
By Balance *brought down—as per Balance Sheet, page 588* .	£413,103	13	10

STRUCTURE, FURNITURE, FITTINGS AND MACHINERY.

	£	s.	d.
By Balance—*per last account*	6,883	7	4
,, Institution Revenue Account—Amount provided for the year—*per page 592*	1,000	0	0
,, Interest on Investments	229	19	4
,, Income Tax Refunded	28	1	6
	£8,141	8	2
By Balance *brought down—as per Balance Sheet, page 588*. .	£6,028	17	7

ACCOUNT

	£	s.	d.
By Institution Revenue Account—Amount provided for the year—*per page 592*	10,000	0	0
,, Balance, *carried down* (being Excess of Expenditure over Provision)	2,414	12	3
	£12,414	12	3

INSTITUTION REVENUE ACCOUNT

EXPENDITURE.

1934–35						
£		£	s.	d.	£	s.
7,356	To House and Establishment Charges—					
	Rates, Taxes and Insurance	5,843	18	8		
	Electric Lighting and Power, Water-Supply, Warming, Ventilating and Telephone	830	19	6		
	Cleaning and Household Expenses	1,094	17	0		
	Refreshments and Assistance at Meetings	160	12	3		
					7,930	7
1,000	,, Repairs and Renewals Reserve—					
	Amount provided for the year, *per page 591*	..			1,000	0
10,853	,, Salaries, Wages and Retiring Allowances—					
	Salaries	3,362	10	0		
	Retiring Allowances	1,605	0	0		
	Clerks, Messengers and Housekeeper	5,271	16	7		
					10,239	6
1,224	,, Premiums on Policies for Staff Pensions—					
	Portion paid by the Institution	..			1,383	7
1,786	,, Stationery, Postages, etc.—					
	Stationery and Printing	1,160	18	7		
	Postages, Telegrams and Parcels	929	9	9		
					2,090	8
14,888	,, Publications Account—					
	Amount provided for the year, *per page 591*	..			10,000	0
—	,, Research Reserve—					
	Amount provided for the year, *per page 597*	..			1,000	0
1,427	,, Library—					
	Books and Periodicals	470	19	1		
	Binding	149	4	10		
	Loan Library Catalogue	89	17	8		
	Clerical Pay and Pensions Premiums	869	0	11		
					1,579	2
3,492	,, Examination Expenses—					
	Examiners, Printing and General	1,899	11	4		
	Salaries, Clerical Pay and Pensions Premiums	1,465	7	3		
	Postages	118	8	0		
					3,483	6
1,202	,, Conversazione and Annual Dinner	..			1,255	0
49	,, Diplomas and Medals—					
	Diplomas	..			41	1
1,083	,, Local Associations—					
	Grants to Local Associations, etc.	..			1,178	3
100	,, Contributions towards Advisory Committees in the Dominions	..			27	14
157	,, Grants and Contributions—					
	Wm. Froude Laboratory Tank Research Fund	100	0	0		
	Iron and Steel Institute Welding Symposium	50	0	0		
	Engineering Joint Council	12	10	0		
	Westminster Hospital	10	10	0		
	Parliamentary Science Committee	10	10	0		
	World Power Conference	3	3	0		
					186	13
	,, Address to the Société Belge	..			6	6
221	,, Legal and other Professional Charges—					
	Legal Charges	265	10	1		
	Audit and Accountancy Fees	446	5	0		
	Engineers' and Surveyors' Fees	60	15	0		
					772	10
	,, Engineering Quantities Report	..			41	5
	,, Floods Report	..			12	1
	,, Silver Jubilee Decorations and Illuminations	..			157	11
305	,, Interest on Loan	..			234	6
					£42,618	13
2,482 (Dr.)	,, Balance, being Excess of Income over Expenditure for the Year carried to General and Contingency Reserve, *page 588*	..			1.296	19
£42,754					£43,915	12

FROM 1ST APRIL, 1935, TO 31ST MARCH, 1936.

INCOME.

	£ s. d.	£ s. d.	1934–35 £
By SUBSCRIPTIONS APPLICABLE TO THE FINANCIAL YEAR 1935–1936	..	31,991 18 4	31,421
,, ENTRANCE FEES	..	6,039 4 0	4,884
,, INTEREST, DIVIDENDS, ETC.—			
On Institution Investments	2,052 4 0		
On Deposit and Current Accounts	81 12 9		
Income Tax refunded for the year 1934–5	203 0 9		
		2,336 17 6	2,314
,, EXAMINATION FEES	..	3,290 9 0	3,691
,, LIBRARY FUND DONATIONS	..	49 11 6	36
,, RENT OF 1, GREAT GEORGE STREET	..	207 12 3	162
		£43,915 12 7	42,754

TRUST

CAPITAL ACCOUNTS AND INVESTMENT THEREOF AND INVESTMENT

Capital Accounts.		Investments. Capital.	Unexpended Income.
£ s. d.		£ s. d.	£ s. d
8,038 9 4	TELFORD FUND.		
	£8,738 13s. 0d. 2½% Consols	7,988 9 4	
	£50 16s. 11d. 3½% War Loan	50 0 0	
270 0 0	MANBY DONATION.		
	£250 London & North-Eastern Railway 4% 2nd Guaranteed Stock . .	270 0 0	
6,337 12 4	MILLER FUND.		
	£5,129 17s. 5d. 2½% Consols	4,850 2 4	
	£1,513 15s. 9d. 3½% War Loan	1,487 10 0	
500 0 0	HOWARD BEQUEST.		
	£352 11s. 5d. 2½% Consols . Cost of Medal Die . . .	500 0 0	
600 0 0	TREVITHICK MEMORIAL.		
	£103 2½% Consols . . .	100 0 0	
	£506 5s. 7d. 3½% Conversion Loan 1961	500 0 0	
540 0 0	CRAMPTON BEQUEST.		
	£512 15s. 11d. 2½% Consols	500 0 0	
	£40 13s. 7d. 3½% War Loan	40 0 0	
1,234 14 0	JAMES FORREST LECTURE AND MEDAL FUND.		
	£465 Southern Railway 4% Debenture Stock . . .	604 14 0	
	£667 5s. 8d. 3½% War Loan	630 0 0	
1,530 18 0	PALMER SCHOLARSHIP.		
	£1,496 6s. 1d. Metropolitan 3% Consolidated Stock .	1,430 18 0	
	£100 9s. 8d. 3½% War Loan	100 0 0	
1,080 0 0	JOHN BAYLISS BEQUEST.		
	£1,013 17s. 10d. London County 3% Stock . . .	1,000 0 0	
	£80 7s. 10d. 3½% War Loan	80 0 0	
1,318 11 8	THE INDIAN FUND.		
	£1,353 4s. 2d. 2½% Consols .	1,148 11 8	
	£171 13s. 3d. 3½% War Loan	170 0 0	
1,000 0 0	VERNON-HARCOURT BEQUEST.		
	£1,082 9s. 10d. London County 3% Stock . .	1,000 0 0	
22,450 5 4	*Carried forward* . .	22,450 5 4	

FUNDS.

OF UNEXPENDED INCOME AT 31ST MARCH, 1936.

Capital Accounts.		Investments. Capital.	Investments. Unexpended Income.
£ s. d.		£ s. d.	£ s. d.
22,450 5 4	*Brought forward* . . .	22,450 5 4	
1,300 0 0	WEBB BEQUEST.		
	£1,055 7s. 2d. Metropolitan Water Board 3% "B" Stock	1,000 0 0	
	£303 16s. 2d. 3½% War Loan	300 0 0	
2,733 1 10	WILLIAM LINDLEY FUND.		
	£1,214 London Midland & Scottish Railway 4% Debenture Stock . . .	1,584 16 8	
	£1,109 2s. 9d. 4½% India Stock, 1950–1955 . . .	1,005 15 2	
	£151 9s. 7d. 3½% War Loan	142 10 0	
725 0 0	KELVIN MEDAL FUND.		
	£757 18s. 11d. 3½% War Loan	725 0 0	
4,250 0 0	CHARLES HAWKSLEY BEQUEST.		
	£955 Metropolitan Water Board 3% "B" Stock .	573 0 0	
	£500 South Essex Waterworks 5% Preference Stock	435 0 0	
	£70 5% Sheffield Corporation Water Annuities . . . £40 4% Sheffield Corporation Water Annuities . .	1,992 0 0	
	£1,257 17s. 4d. 3½% War Loan	1,250 0 0	
1,101 6 5	COOPERS HILL WAR MEMORIAL.		
	£1,239 15s. 0d. 3½% War Loan	1,101 6 5	
	£215 12s. 3d. 3½% War Loan	..	214 8 2
3,570 4 0	C. C. LINDSAY CIVIL ENGINEERING SCHOLARSHIP FUND.		
	£3,500 3½% War Loan . .	3,570 4 0	
320 0 0	BAKER MEDAL FUND. . .		
	£290 5s. 8d. London County Cons. 4½% Stock 1945–85	320 0 0	
36,449 17 7	*As per Balance Sheet, page 589*	*36,449 17 7	*214 8 2

NOTE.—* *The value of these Investments at ruling prices on 31st March, 1936, amounted approximately to £37,526 and £230 respectively.*

TRUST FUNDS INCOME ACCOUNTS FROM

Trust Fund.	Balance at 1st April, 1935. £	s.	d.
Telford Fund	45	18	7
Manby Fund	26	2	9
Miller Fund	98	5	4
Howard Bequest	36	6	2
Trevithick Memorial	23	11	3
Crampton Bequest	6	8	0
James Forrest Lecture and Medal Fund	32	0	4
Palmer Scholarship Fund	21	5	9
John Bayliss Bequest	43	8	4
Indian Fund	20	0	10
Vernon-Harcourt Bequest	91	14	2
Webb Bequest	109	5	6
William Lindley Fund	399	5	8
Kelvin Medal Fund	95	2	4
Charles Hawksley Bequest	161	10	0
Coopers Hill War Memorial Fund	315	16	4
C.C. Lindsay Civil Engineering Scholarship Fund	161	14	10
Baker Medal Fund	4	7	11
Totals	1,692	4	1

COMMITTEE ON THE DETERIORATION OF

Account from 1st APRIL, 1935,

	£	s.	d.
To Amount paid on behalf of or to the Committee during the year to 31st March, 1936	485	13	2
,, Balance *carried down*	2,121	18	3
	£2,607	11	5

RESEARCH

	£	s.	d.
By Balance *carried down*	1,200	0	0
	£1,200	0	0

1st APRIL, 1935, TO 31st MARCH, 1936.

Income: Including Income Tax refunded for the year 1934–1935.			Expenditure on Scholarships, Prizes, Lectures, etc.			Balance at 31st March, 1936.		
£	s.	d.	£	s.	d.	£	s.	d.
220	9	4	214	5	11	52	2	0
10	2	5	16	0	0	20	5	2
181	13	10	102	13	3	177	5	11
8	19	8	0	0	0	45	5	10
20	8	2	30	0	5	13	19	0
14	5	5	16	10	6	4	2	11
42	2	2	47	14	6	26	8	0
48	9	11	40	0	0	29	15	8
33	8	8	30	0	0	46	17	0
39	18	4	33	0	0	26	19	2
32	17	11	0	0	0	124	12	1
42	16	1	0	0	0	152	1	7
105	12	9	25	0	0	479	18	5
26	19	4	51	2	0	70	19	8
208	9	1	258	13	10	111	5	3
51	8	3	44	17	9	322	6	10
123	4	11	67	10	0	217	9	9
13	1	7	0	0	0	17	9	6
1,224	7	10	977	8	2	†1,939	3	9
						As per Balance Sheet p. 588.		

† Of which £214 8*s*. 2*d*. is invested (*see page 595*).

STRUCTURES EXPOSED TO SEA ACTION.

TO 31st March, 1936.

	£	s.	d.
By Balance, as per last Account	1,967	0	10
,, Subscriptions	630	4	2
,, Interest on Deposit	10	6	5
	£2,607	11	5
,, Balance *brought down as per Balance Sheet, page 588*	£2,121	18	3

RESERVE

	£	s.	d.
By Contributions from other Bodies	200	0	0
,, Institution Revenue Account—Amount provided for the year—*per page 592*	1,000	0	0
	£1,200	0	0
,, Balance *brought down as per Balance Sheet, page 588*	£1,200	0	0

INSTITUTION INVESTMENTS AT 31st MARCH, 1936 (INCLUDING THOSE HELD IN RESPECT OF REPAIRS AND RENEWALS RESERVE) AT COST.

£	s.	d.		£	s.	d.
3,000	0	0	Metropolitan Water Board 3% "B" Stock .	2,958	16	0
6,000	0	0	London and North Eastern Railway 4% Debenture Stock	7,749	18	3
6,000	0	0	London Midland and Scottish Railway 4% Debenture Stock	7,452	14	8
2,545	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	1,976	7	10
38,650	18	7	3½% War Loan	38,453	10	4
2,164	12	6	5% Conversion Loan 1944–1964 "A" . . .	2,206	15	3
2,720	5	5	London Passenger Transport Board 4½% "A" Stock	3,327	9	3
3,809	0	2	3½% War Loan	3,824	8	6
452	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	351	3	7
1,242	5	6	5% Conversion Loan 1944–1964 "A" . . .	1,296	18	1
989	14	7	London Passenger Transport Board 4½% "A" Stock	1,210	12	11
			As per Balance Sheet, page 589 .	£70,808	14	8

NOTE.—*The value of these Investments at ruling prices on 31st March, 1936, amounted approximately to £73,007.*

W. A. P. TAIT LEGACY.

£	s.	d.		£	s.	d.
505	19	9	3½% War Loan	514	11	8

NOTE.—*The value of this investment at ruling prices on 31st March, 1936 amounted approximately to £540.*

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Sub-Committee on Earthing to Metal Water-Pipes aud Mains.

THE Electricity Acts, 1882 and 1933, and the Electricity Supply Regulations arising therefrom, provide for the necessity of earthing a direct- or alternating-current system of transmission and distribution, and also the metal-work enclosing or supporting electrical apparatus. The Factory Acts provide for similar and stringent regulations for the safety of the operative in factories.

Telegraph, telephone and wireless apparatus also necessitate in most cases a connection to earth, and it has been the custom for years in domestic and other premises to earth electrical apparatus, conduits, and wireless installations to the incoming water service where such service exists. This forms the cheapest and best method and usually provides for a path to earth of low resistance.

Many cases have arisen where corrosion to water-mains has been attributed to and, in some cases, proved to be due to such earthing, and there have been instances where electric shocks have been incurred during the repair of water-mains. While it is generally acknowledged that continuous current causes electrolytic corrosion where it leaves a metal conductor or pipe to earth, there is some doubt about the action of alternating current. In view of the importance of the subject to both electrical engineers and water engineers, a Sub-Committee on Earthing to Metal Water-Pipes and Mains has been set up with the approval and collaboration of the Institution of Electrical Engineers, the Institution of Water Engineers and the Water Associations.

The following are the terms of reference :—

To explore the problem of possible injury to metal water-pipes and mains through the earthing thereto of electrical installations, particularly in relation to alternating currents, with a view

(*a*) to investigating the existence and extent of such injury, research being carried out if necessary,

(*b*) to obtaining mutual agreement on the conditions under which earthing connections to water-pipes and mains might be made, and

(*c*) to formulating, if necessary, a set of regulations in respect thereof.

The composition of the Sub-Committee is as follows :—

S. B. Donkin (*Chairman*).
H. J. F. Gourley, M.Eng.
E. F. Law.

Ll. B. Atkinson. P. Dunsheath, O.B.E., M.A., D.Sc. F. W. Purse. P. J. Ridd.	Representing the Institution of Electrical Engineers.
H. F. Cronin, M.C., B.Sc. B. W. Davies.	Representing the Institution of Water Engineers.
R. W. James. F. J. Dixon.	Representing the British Waterworks Association.

J. D. K. Restler, O.B.E. Representing the Water Companies Association.

H. W. Swann, of the Home Office.

R. G. Hetherington, C.B., O.B.E., M.A., of the Ministry of Health.

Sub-Committee on Fish-Passes.

Dams and weirs in rivers form obstructions to the passage of fish unless some form of pass is provided. Whilst the maintenance of the life of fish which swim to the upper reaches of rivers is of national importance, the industrial conditions of to-day require that the water be controlled to the best advantage of the user, and frequently the requirements of the two interests are in conflict. The Council have accordingly formed a sub-committee under the Institution Research Committee to investigate the problems arising out of the design of fish-passes.

Several forms of fish-pass have been devised, mostly consisting of a series of pools in the form of a cascade. In the past the fishing interests have usually required a large quantity of water for the pass, but recent experience shows that greater efficiency is obtained with smaller flows. It is desirable, therefore, that research should be made to determine, if possible, the most efficient form of pass to meet all interests. The problems to be studied involve hydraulic model experiments which are being carried out under the direction of Dr. C. M. White at the City and Guilds (Engineering) College. The research will also include a study of existing forms of fish-pass and of scale-model experiments checked by large-scale tests to correlate results with practical conditions, and is estimated to take about a year.

Financial assistance to this research is being given by The Institution, the Worshipful Company of Fishmongers, the Development Commissioners, and by other interested bodies.

The personnel of the Sub-Committee is as follows :—

W. T. Halcrow (*Chairman*).
Rustat Blake, M.A.
David Lyell, C.M.G., C.B.E., D.S.O.
A. M. MacTaggart.
W. F. H. Creber. Representing the British Waterworks Association.
T. E. Hawksley, B.A. Representing the Institution of Water Engineers.
C. M. White, B.Sc., Ph.D.
T. E. Pryce-Tannatt. } Representing the Ministry of Agriculture and
J. C. A. Roseveare. } Fisheries.
W. J. M. Menzies. Representing the Fishery Board for Scotland.
R. Beddington. Representing the National Association of Fishery Boards.
W. Malloch, B.Sc. Representing the Association of Scottish District Salmon Fishery Boards.

Joint Committee on Simple-Span Girder Bridges.

A Joint Committee of the Institutions of Civil Engineers and Structural Engineers has been set up to draw up a code of practice and advice in connection with the design and construction of simple-span girder bridges. The following is the composition of the Committee :—

Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S. (*Chairman*).
David Anderson, LL.D., B.Sc.
C. J. Brown, C.B.E.
G. S. Gough, M.A.
J. F. Pain, M.C., B.Sc.
Professor A. J. S. Pippard, M.B.E., D.Sc.
E. S. Andrews, B.Sc. }
H. P. Budgen, Ph.D., M.Sc. } Representing the Institution of
Oscar Faber, O.B.E., D.Sc. } Structural Engineers.
Professor Joseph Husband, M.Eng. }

The decision to appoint this Committee arose out of the consideration of the recently completed revision by the British Standards Institution of B.S. Specification No. 153 (Girder Bridges, Part 3, Loads and Stresses). In dealing with a matter of this kind by specification, there are many points at which freedom must be given to the designing engineer, but in which engineers would be glad to have advice in the light of the latest scientific work on the subject. Such a case arises, for instance, in dealing with the questions of impact, lurching of locomotives and other dynamic effects, where the British Standards Institution revision committee found that the provisional formula for impact allowance on railway bridges set out in the 1923 issue of the specification could not be allowed to remain, and that agreement could not be reached on any one method which could be used in its place.

In this and other instances it is believed by the two Institutions

that authoritative guidance would be welcomed by engineers concerned in the design and construction of girder bridges. The scope of the Committee's work is limited to simple-span girder bridges, as it is felt that no general code can be formulated of universal application to more complex structures.

It may be added that the British Standards Institution have welcomed the appointment of this Committee, which is fortunate in including in its membership Mr. C. J. Brown, the Chairman of the Committee of the British Standards Institution which undertook the revision of Specification No. 153, Part 3.

THE STEEL STRUCTURES RESEARCH COMMITTEE.

Recommendations for rules for the design of multi-storey steel-framed buildings in a more exact manner than is implied in existing constructional codes, resulting from 6½ years of research, are embodied in the Final Report of the Steel Structures Research Committee of the Department of Scientific and Industrial Research, which has been published by H.M. Stationery Office, price 12*s*. 6*d*. The draft rules for design, reprinted from the full report, are obtainable at a cost of 6*d*. A review of this Report has not been included as the main results of this research were set forth in Professor J. F. Baker's recent Paper.[1]

NOTES ON RESEARCH PUBLICATIONS.

Engineering Materials: Properties and Testing.

Timber.

The effect of rate of growth upon the specific gravity and strength of Douglas fir is stressed in *Circular 44, Department of the Interior, Canada* (*Forest Service*), and in *Circular 45* an account is given of tests on entire telephone poles of Douglas fir (*Eng. Abs.* **70,** *No. 30*). The Forest Products Research Laboratory of the Department of Scientific and Industrial Research has issued *Research Record No. 11* on the properties of home-grown oak.

Cement and Concrete.

The cone method for determining the absorption of water by sand is described in *Am. Soc. Test. Mat. Reprint No. 60* (*Eng. Abs.* **70,** *No. 14*).

[1] "The Rational Design of Steel Building Frames." Journal Inst. C.E., vol. 3 (1935–36), p. 127. (June, 1936).

The figure in heavy type is the number of the Volume; the figure in brackets the number of the Part; and that in italic type the number of the Page.

An account of research on cracking in reinforced concrete at the Building Research Station is given in *Structural Engineer*, **14**, *298*, and on *p. 321* a method for the estimation of the compressive strength of concrete in the field is described, in which the indentation by a steel ball shot from a pistol is measured. The behaviour of concrete under various storage conditions as determined by means of measurements of changes in length is dealt with in *Beton u. Eisen*, **35**, *169*. The effect of testing speed on strength and elastic properties of concrete is discussed in *Am. Soc. Test. Mat.*, *Preprint No. 59* (*Eng. Abs.* **70**, *No. 16*). In *Zement*, **25**, *233*, the compressive and impact strength and abrasion resistance of concrete are dealt with. Determination of the shrinkage stresses exerted on a body embedded in a cement mass is given in *Compt. Rend.*, **202**, *1153*. The *Proceedings of the Physical Society*, **48**, *498*, contains a paper on the thermal constants of setting concrete.

Articles dealing with special cements are: Experience with special cement, *Zeit. angew. Chemie.*, **49**, *85*: Trass cement—blast-furnace slag cement, *Bautech.*, **14**, *183*: Calcium chloride admixtures and the early strength of cement, *Zement*, **25**, *132*.

The following deal with deterioration of concrete: Resistance of cement to attack by water containing calcium sulphate as determined by Anstett's test, *XV^me Congrés de Chimie Industrielle* (Brussels, 1935), *Compt. Rend.*, *p. 134*, and in the same journal, *p. 577*, New experiments on the attack on mortars by pure water: The deterioration of concrete owing to chemical attack, a description of work carried out at the Building Research Station, *Inst. San. Engineers J.*, **40**, *185*: Methods for the rapid determination of the resistance of cement concrete to the action of mineralized waters, *Scientific Research Inst. Hydrotechnics, Leningrad, Trans.* **16**, *148* (*Eng. Abs.* **69**, *No. 15*), and in **17**, *197*, an article on the action of weak solutions (similar to natural sulphate-containing waters) upon various types of cements: Determining the resistance of Portland cement to sulphate waters, *Ind. Eng. Chem. Analyt. Edn.*, **8**, *263*: Barium carbonate as means of protecting concrete against sulphate waters, *Ton-Industrie Zeit.*, **60**, *443*.

Metals.

The theory of alloy structures, a description of the present position of research by means of X-ray crystal analysis, is given in *Roy. Aeronaut. Soc. J.*, **40**, *409*, and on *p. 586* is an article on the strength of metals in the light of modern physics. A paper entitled: A new attack upon the problem of fatigue of metals using X-ray methods

of precision and describing work carried out at the National Physical Laboratory is given in *Roy. Soc. Proc. Series A*, **154,** *510*. A description of slow-bend and impact tests of notched bars at low temperatures is given in *Am. Soc. Test. Mat., Preprint No. 30* (*Eng. Abs.* **70,** *No. 18*), and *Preprint No. 28* deals with high-velocity tension-impact tests. In *Inst. Mech. Eng. Proc.*, **131,** the following papers are given: *p. 3,* The strength of metals under combined alternating stresses: *p. 131,* The utilization of creep test data in engineering design. The influence of thickness upon the strength of metals and alloys is discussed in *Ver. deu. Ing.*, **80,** *933* (*Eng. Abs.* **70,** *No. 26*). Publications dealing with various metals are: Strength and elastic properties of cast iron, *Iowa Univ. Eng. Expt. Stn. Bull. 127*: Internal stresses and their effect on the fatigue resistance of spring steels, Further experiments on the effect of surface conditions on the fatigue resistance of steels, and The behaviour of five cast irons in relation to creep and growth at elevated temperatures, papers read before the Iron and Steel Institute on the 7th May, 1936: The influence of hardening and age after hardening upon some properties of mild steel is dealt with in *Chimie et Ind.*, **35,** *27* (*Eng. Abs.* **69,** *No. 32*). The correlation between impact and static torsion tests for carbon steels at low and high temperatures is discussed in *Technology Reports of the Tôhoku Imperial University, Japan*, **12,** *63*. In *Roy. Soc. Proc., Series A*, **156,** *383*, is a paper on acoustic studies of some non-transforming and transforming special steels at low temperatures. Wire-rope research in relation to colliery practice is dealt with in *Inst. Mining Engineers J.*, **91,** *196*. The influence of temperature on the severity of corrosion fatigue is discussed in *Engineering* **141,** *495*.

Corrosion is dealt with in the following publications: Damage caused to metal by water impact, *Ver. deu. Ing.*, **80,** *863* (*Eng. Abs.* **70,** *No. 83*): The corrosion of underground gas-mains and services, *Gas J.*, **213,** *309*: Soil-corrosion studies, 1934. Rates of loss of weight and pitting of ferrous specimens, *U.S. Nat. Bur. Stand. J. Research*, **16,** *431* (*Eng. Abs.* **70,** *No. 25*), a report on a comprehensive research extending over 14 years.

Other Materials.

A study of the strength of flat glass under uniform load is described in *Am. Ceram. Soc. Bull.*, **15,** *243*: a comparison of statically and dynamically determined Young's modulus of rocks, *Proc. Nat. Academy of Sciences*, **22,** *81*: Change of penetration with temperature of various asphalts, *Ind. Eng. Chem. Analyt. Edn.*, **8,** *157*, and on *p. 185* is an article on the determination of flow properties of bituminous materials.

Engineering Materials: Production, Manufacture and Preservation.

Timber.

The practice of wood bending is discussed in *Forest Products Research Record No. 10* of the Department of Scientific and Industrial Research.

Concrete.

The following papers on the production and preservation of concrete appear in *Amer. Concr. Inst. J.*, **7**: *p. 593*, Study of the durability of concrete: *p. 609*, Alternate heating and cooling of mortar: *p. 621*, Studies of high-pressure steam curing of concrete slabs and beams: *p. 641*, Factors of workability of Portland-cement concrete. The electric heating of concrete during winter construction is described in *Schw. Bauz.*, **107**, *69*. (*Eng. Abs.* **68,** *No. 33.*) The subject of concrete vibration is dealt with in *Ann. P. et C.*, **106**-*i*, *338* (*Eng. Abs.* **6 9,** *No. 29*). A review of the results of investigations on high-pressure steam curing is given in *Concrete*, **44,** *19 and 26*.

Metals.

A method of developing a dense protective coating on aluminium by an electrochemical process is described in *Zeit. angew. Chemie*, **49,** *493*.

Mass Structures.

Researches dealing with earthworks include: a study of pressure-distribution in a non-continuous subsoil, *Sci. et Ind.* (*Travaux*), **20,** *226* (*Eng. Abs.* **70,** *No. 50*): a critical survey of the Rankine and Coulomb theories of earth pressure, *Boston Soc. Civ. Engineers J.*, **23,** *71*: Internal friction in soil, *Teknisk Tidskrift*, **66,** *21*, and in the same volume Friction and cohesion in soil, p. *26*: Internal stability of granular materials, *Am. Soc. Test. Mat.*, *Preprint No. 80*.

Hydrostatic uplift is the subject of the following papers: Electrical method of investigating uplift pressure under dams and weirs, *Punjab Irrigation, Inst. Mem.*, **5,** *No. 4* (*Eng. Abs.* **70,** *No. 3*), and in the same volume, *No. 5* (*Eng. Abs.* **70,** *No. 182*), Flotation gradient for flow of water through porous strata and its bearing on the stability of foundations: The seepage flux under dams of extended base width and under cofferdams resting on permeable strata of finite thickness is discussed in *Physics*, **7,** *116*, and simple tests to determine hydrostatic uplift in *Eng. News-Record*, **116,** *872* (*Eng. Abs.* **70,** *No. 54*).

Framed Structures.

Research on structural elements: an analysis of finite strain in elastic problems, in which account is taken of 2nd order terms in the components of strain, *Proc. Roy. Soc., Series A*, **156,** *171,* and in the same journal, *p. 518,* a paper on an experimental method for the solution of plane stress problems: Calculation of the deflexion of rectangular plates by simple series, *Ann. Trav. Publics Belg.*, **37,** *291* (*Eng. Abs.* **70,** *No. 58*): Stresses in a notched plate under tension, *Phil. Mag.*, **21,** *765*: Tests on the stress-bearing width of buckled plates, *Luftfahrt*, **13,** *214* (*Eng. Abs.* **70,** *No. 59*): a paper on Castigliano's principle of minimum strain energy, *Roy. Soc. Proc., Series A*, **154,** *4,* and in the same journal, *p. 430,* the equilibrium and elastic stability of a thin twisted strip: Buckling in a curved element, *Zeitschrift für angewandte Mathematik und Mechanik*, **16,** *49.*

In connection with the stresses in structural frames, an analysis of the Vierendeel truss is given in *Proc. Am. Soc. C.E.*, **62,** *833*: an analysis of the stresses in rigid suspension bridges in *Sc. et Ind.* (*Travaux*), **41,** *218*: a simple approximate method of solution for the design of statically indeterminate trusses, in *Boston Soc. Civ. Engineers J.*, **23,** *57*: an account of tests and the design of steel wind bents for tall buildings, *Ohio State Univ. Eng. Expt. Sta., Bulletin No. 93*: a simplified method of designing multi-storey frames subjected to lateral loads is given in *Eng. News-Record*, **116,** *922.*

The following tests on structural members are described in *U.S. Nat. Bur. Standards J. Research*, **16,** *265* (*Eng. Abs.* **70,** *No. 75*), Some tests of steel columns encased in concrete, which show an increase in strength over the unencased column of 50 per cent. at the yield-point: *p. 595* (*Eng. Abs.* **70,** *No. 60*) Tests of 8 large H-shaped columns fabricated from C–Mn steel: *p. 627* (*Eng. Abs.* **70,** *No. 70*), Tests of steel chord members for the Bayonne bridge. An investigation of the use of bamboo reinforcement in concrete structures is described in *Bauing.*, **17,** *17* (*Eng. Abs.* **68,** *No. 75*).

Constructional Operations and Methods.

In *Scientific Research Inst. Hydrotechnics, Leningrad, Trans.* **17,** *66,* is described a method of construction of dams and other structures by dumping stones into flowing water. An article on research on shuttering is given in *Wirtschaftl. Bauer*, *1935* (*14*), *5.* In *Ann. P. et C.*, **106-***i*, *423,* the construction of prestressed reinforced-concrete pressure conduits for a hydroelectric power scheme is described.

Transformation, Transmission and Distribution of Energy.

Research on the pressure-drop in steam pipe-lines is described in *Archiv Wärme*, **17,** *101* (*Eng. Abs.* **69,** *No. 100*). A description of a new hydrogen- and water-cooled turbo-generator is given in *Elec. W.*, **106,** *1017, and 1086* (*Eng. Abs.* **69,** *No. 104*). A novel principle of propulsion by thermodynamic tubes is explained in *Compt. Rend.*, **202,** *52* (*Eng. Abs.* **69,** *No. 107*). The effects of overstrain on closely-coiled helical springs and variations of the number of active coils with load is described in *Bull. Virginia Poly. Inst.*, **29,** *8* (*Eng. Abs.* **70,** *No. 23*). A paper on the alternating-current resistance of tubular conductors describing research at the National Physical Laboratory is given in *Inst. Elec. Engineers J.*, **78,** *580.*

Mechanical Processes, Appliances and Apparatus.

Research in connection with welding is dealt with in Temperature stresses in flat rectangular plates and in thin cylindrical tubes, *Franklin Inst. J.*, **222,** *149*, and Physico-chemical phenomena of the transfer of metal during welding in *Am. Welding Soc.*, **15,** *26* (*Eng. Abs.* **69,** *No. 135*). Results of experience in connection with the deposition of concrete by pump and pipe-line are summarized in *Am. Concrete Inst. J.*, **7,** *333* (*Eng. Abs.* **69,** *No. 140*).

Important researches on the analysis of commercial lubricating oils by physical methods are summarized in *Lubricating Research Technical Paper No. 1* of the Department of Scientific and Industrial Research. The breakdown of the lubricating film is studied and shown to be dependent on the chemical composition rather than the physical properties of the oil.

Specialized Engineering Practice.

Transport.

The work of the German Road Research Society on the effect of surface crazing on the durability of concrete road slabs is described in *Mitteilungen der Forschungsgesellschaft für das Strassenwesen* (*Berlin*) *1936* (*8*), *68*, and a study of the dynamic contact force between tyre and road is given in *Teknisk Tidskrift*, **66,** *9.*

The following researches deal with railways: Behaviour of railway vehicles on curves, *Schw. Bauz.*, **107,** *178* (*Eng. Abs.* **69,** *No. 152*): Development of draught gears for American freight cars, *Purdue University Engineering Bulletin No. 54*, and Impact of railway vehicles in relation to buffer resistance, *Inst. Locomotive Engineers J.*, **26,** *209.* Researches on water transport include: Dimensional analysis of model propeller tests, *Am. Soc. Nav. Eng.*, **48,** *147* (*Eng.

Abs. **70,** *No. 132*): The Bénard-Kármán turbulence behind an obstacle moving in a canal, *Comptes Rend.*, **202,** *1021* (*Eng. Abs.* **69,** *No. 215*); Propeller vibration, *Trans. Soc. Naval Architects and Marine Engineers*, **43,** *252* (*Eng. Abs.* **70,** *No. 166*), and in the same journal, *p. 286*, New studies of ship motion: and the following articles in *Scientific Research Institute of Hydrotechnics, Leningrad, Trans.* **17**: *p. 76*, Some considerations regarding the length of hydraulic jump: *p. 127*, Transportation of bedload: *p. 145*, Theoretical principles of designing transit stream flows: *p. 158*, Some considerations of steady slowly-varied flow of liquids in regularly shaped non-prismatic channels.

Aeronautical research is dealt with in the following: Theory of the shrouded propeller, *Werft*, **17,** *221 and 239* (*Eng. Abs.* **70,** *No. 130*): Study of the air-flow around models by the soap bubble method, *Aéronautique*, **18,** *60* (*Eng. Abs.* **70,** *No. 171*): and the following articles in the *Roy. Aeronautical Soc. J.*, **40**: *p. 420*, The stressing of rigid-jointed frames: *p. 483*, Air-screw development: *p. 663*, Elastically encastered struts: *p. 681*, Theory of the slat in a two-dimensional flow. The following *Aeronautical Research Committee Reports and Memoranda* have been noted: *No. 1602*, Influence of a uniform jet on the lift of an aerofoil: *No. 1675*, Graphical method of calculating performance of airscrew: *No. 1678*, Laminar boundary layer on the surface of a sphere in a uniform stream: *No. 1682*, Effect of weight on take-off and landing: *No. 1685*, Effect of variation of aileron inertia and damping on flexural-aileron flutter of a typical cantilever wing: *No. 1686*, Flutter experiments on a model wing fitted with a dead-centre aileron control: *No. 1687*, Performance and longitudinal stability of a single-engined high-wing monoplane: *No. 1690*, Stressing of aeroplane wings due to symmetrical gusts. In *The Engineer*, **162,** *120*, *Engineering* **142,** *129*, is a description of the direct take-off type autogiro. The National Advisory Committee for Aeronautics (U.S.A.) has issued the following reports: *No. 546* (*Eng. Abs.* **69,** *No. 161*), Effect of turbulence on the drag of flat plates: *No. 548* (*Eng. Abs.* **69,** *No. 164*), Effect of tip shape and dihedral on lateral stability characteristics: *No. 555*, Air-flow around finned cylinders. In *Luftfahrt*, **13,** *85* (*Eng. Abs.* **69,** *No. 162*), is a paper on the lift of a bent plane plate; *p. 103* (*Eng. Abs.* **69,** *No. 163*), Calculation of airscrews of small thrust coefficients and advance constants, and *p. 111* (*Eng. Abs.* **69,** *No. 166*), Wing-propelled aircraft.

Water-Supply.

The first annual report of the Inland Water Survey Committee, 1935–6, has been published. Flow of water in a channel of compound

cross-section is dealt with in *Ann. P. et C.*, **106**-*i, 445* (*Eng. Abs.* **70,** *No. 164*).

Mining.

The 14th annual report of the Safety in Mines Research Board, 1935, has been published. *Safety in Mines Research Paper No. 96*, deals with the inflammation of coal-dust and the effect of carbon dioxide and combined water in the dust. In *Trans. Inst. Mining Engineers*, **91,** *104*, an electro-magnetic test of wire ropes is described, and on *p. 127* recent developments in blasting in coal mines are dealt with.

Heating, Ventilating and Acoustics.

The pressure losses at elbows in ventilating ducts are discussed in *Heating, Piping*, **8,** *365 and 427*. The sound-absorption value of Portland-cement concrete is dealt with in *Am. Conc. Inst. J.*, **7,** *659*. A paper on noise is given in *Proc. Inst. Mech. E.*, **130,** *479*. Experiments to determine the heat and sound insulation of different types of floor construction are dealt with in *Schw. Bauz.*, **107,** *19*.

Miscellaneous.

In *Roy. Soc. Proc., Series A*, **155**, the following papers have been noted : *p. 570*, A note on the measurement of total head and static pressure in a turbulent stream : *p. 576*, On the static pressure in fully-developed turbulent flow : and in volume **156,** *p. 307*, Statistical theory of turbulence. In *Proc. Cambridge Philosophical Soc.*, **32,** are the following papers : *p. 380*, The mean value of the fluctuations in pressure and pressure-gradient in a turbulent fluid : *p. 385*, Note on the velocity-distribution in the wake behind a flat plate placed along the stream : and *p. 392*, Boundary-layer growth.

NOTE.

OBITUARY.

KENNETH ALFRED WOLFE BARRY, O.B.E., was born on 16 March, 1879, the second son of the late Sir John Wolfe Barry, K.C.B., LL.D., F.R.S., Past-President Inst. C.E., and died on 1 July, 1936, in London. He went to Winchester in 1892 and from there to Trinity College, Cambridge, where he carried out 2 years' engineering study. In 1899 he was articled to his father's firm for practical training, and subsequently gained varied experience in connection with civil engineering works, including the Surrey Commercial Docks Extension; Port Natal, Durban Harbour; Middlesbrough Dock Extension; the Great Northern, Piccadilly and Brompton Railway; and the Whitechapel and Bow Railway. He then became associated with his father's firm as a partner and was at the time of his death senior partner.

Among his personal appointments outside those held by his firm, he was consulting engineer, jointly with others, to the Bombay Port Trust, the Aden Port Trust, the Southern Punjab Railway, and the Darjeeling and Himalayan Railway. As a member of his firm he was a consulting engineer to the Bengal–Nagpur Railway, the Kowloon–Canton line, and the Tower bridge; and was connected with the construction of the Immingham dock, the Alexandra dock, Newport, Mon., the new Fish dock completed only recently at Grimsby, and numerous other important works.

During the War Mr. Wolfe Barry obtained a commission in the Royal Garrison Artillery, but was seconded for service at H.M. Factory, Gretna (Ministry of Munitions, Department of Explosives Supply), where he later became Assistant Superintendent, and was made O.B.E.

Mr. Wolfe Barry succeeded Mr. Austin Taylor as Chairman of Westminster Hospital in 1928. He had previously been for 16 years a member of the house committee. His great services to the hospital were marked by an industry and devotion that enabled him to triumph over difficulties which at times seemed almost insuperable. He brought forward the rebuilding scheme now in progress, and was largely responsible for the great effort to obtain the preliminary fund of £100,000.

Mr. Wolfe Barry was elected an Associate Member of The Institution in 1905 and was transferred to full Membership in 1913. He was elected a Member of Council in 1935. He became a Member of the Institution of Mechanical Engineers in 1920.

He married Helen Mary, daughter of the late Mr. John Strain, M. Inst. C.E., in 1902, by whom he had three sons and one daughter.

Professor WILLIAM ERNEST DALBY, M.A., B.Sc., F.R.S., was born on 21 December, 1862, and died in London on 25 June, 1936. He commenced his engineering training at the age of fourteen in the Stratford works of the old Great Eastern Railway, and during his apprenticeship he obtained a Whitworth Scholarship in 1883. From 1884 to 1891 he acted as chief assistant to Mr. Bridgewater, of the Permanent Way Department of the London and North-Western Railway at Crewe, and from there he was appointed to assist Sir Alfred Ewing in the development of an engineering department at Cambridge University, where the value of his work was recognized by the University by conferring upon him the M.A. degree, *honoris causa*, in 1894.

He left Cambridge in 1896 to take up the Professorship of Mechanical Engineering and Applied Mathematics at the City and Guilds Technical College at Finsbury in succession to Professor John Perry. He held this position until 1904, when he was appointed to the London University Professorship of Civil and Mechanical Engineering at the Central Technical College (afterwards the City and Guilds (Engineering) College) in succession to Professor W. C. Unwin. He held the Chair until his retirement in 1931, when he was made Emeritus Professor of Engineering. He was Dean of the College from 1906 to 1931.

During the War Professor Dalby did much confidential work for the Admiralty, the War Office and the Air Board. He was a member of the Board of Invention and Research, consulting engineer to the Admiralty Engineering Laboratory, secretary to the sectional committee on engineering of the War Committee of the Royal Society, a member of the expert sub-committee of the Petroleum Executive, and a member of the expert sub-committee of the Gas Traction Committee.

Professor Dalby was elected an Associate Member of The Institution in 1894 and was transferred to full Membership in 1898. His valued services to engineering education and to the engineering profession were marked by his election as a Member of Council in 1925 and as a Vice-President in 1933. The honours he received from The Institution included the George Stephenson Gold Medal in 1906, the Crampton Prize in 1911, and the Howard Quinquennial Medal in 1927. He was elected F.R.S. in 1913 and was a Past Vice-President of the Institution of Mechanical Engineers, and honorary Vice-President of the Institution of Naval Architects. He was the author of well-known text-books and made frequent contributions to the Proceedings of various Institutions. He was a member of the Athenæum Club.

He married Jessie, daughter of Mr. William Holliday, in 1897, by whom he had one daughter.

Sir ARCHIBALD DENNY, Bart., LL.D., was born on 7 February, 1860, and died in London on 29 May, 1936. He was educated at the Academy, Dumbarton, the École Cantonale at Lausanne, and the Royal Naval College, Greenwich, into which he passed after having served a 3 years' apprenticeship at his father's shipyard. He was then for a few months on the staff of Lloyds' Registry at Liverpool and returned to Dumbarton in 1883 to take up a partnership in the family firm. He was almost at once placed in charge of the scientific and technical departments and played an important part in the many improvements introduced by his firm until his retirement. These included the development of cross-Channel steamers and similar craft, and of the use of the turbine engine in commercial shipbuilding.

In recognition of his life of public service a baronetcy was conferred on him in 1913. He was given the honorary degree of LL.D. by the University of Glasgow in 1911 and by the University of Cambridge in 1927.

Sir Archibald was elected a Member of The Institution of Civil Engineers in 1900, a Member of Council in 1914, and Vice-President in 1923. He was obliged to decline nomination to the Presidency in 1928, owing to failing health, but his long association with The Institution was marked by continuous and active interest in its welfare.

He was a Past-President of the Institution of Engineers and Shipbuilders in Scotland, of the Institute of Marine Engineers, and of the Junior Institution of Engineers. He was a Vice-President of the Institution of Naval Architects and Honorary President of the British Corporation for the Registration of Shipping. He was a member of the North-East Coast Institution of Engineers and Shipbuilders. Among other public positions, he was chairman of the Committee on Bulkheads from 1912 to 1915, and of the British Engineering Standards Association, now the British Standards Institution, from 1918 to 1927. He also presided over the Conference on Safety of Life at Sea which was held in 1913–14.

Sir Archibald married in 1885, Margaret, daughter of Mr. John Tulloch, engineer, of Dumbarton, by whom he had five sons and one daughter. He is succeeded in the title by his eldest son, Maurice Edward, born in 1886.

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CORRESPONDENCE

ON

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Paper No. 4989.[1]

"Corrosion of Iron and Steel."

By Sir ROBERT ABBOTT HADFIELD, Bart., D.Sc., D.Met., F.R.S., M. Inst. C.E., and SIDNEY ARTHUR MAIN, B.Sc.

Correspondence.

Mr. WILLIAM BENNETT, of Philadelphia, observed that it was of interest to note how well Halifax came out relatively in the series of tests carried out at that port together with those at Colombo, Auckland, and Plymouth. A statement on p. 10 required some modification or explanation. The Authors said that "A further important discovery of recent times, and one which established a direct and previously-unsuspected effect of corrosive action on the strength of metals, concerns what is known as 'corrosion fatigue.' The weakening of structures due to actual loss of metal through corrosion is obvious, but it was not so clear that corrosion to a degree short of that which causes a reduction in the section of metal can cause a marked reduction in strength, in many cases by as much as ... per cent." That statement made Mr. Bennett think of the many cases of badly-corroded oil-tankers, whose plating was, in addition, by 30 per cent. less in thickness than when the ship had been built; it gave rise to thoughts as to how such oil-tankers managed to survive heavy winter storms, and in many cases even full gales and hurricanes. His impressions of the Paper were: (*a*) The cause (or causes) of corrosion, as evidenced by the Paper, appeared to be more elusive than ever. (*b*) Ordinary mild steel was the basic all-round metal from the engineering standpoint; that was to say, both on account of its satisfactory physical qualities and its good riveting and welding qualities. (*c*) It was, perhaps, a matter for thanks more than for regret, looking at the question purely from the ship-repairers' point of view, that the so-called rustless and stainless steels were not at present producible at a price at all comparable with ordinary mild Mr. Bennett.

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[1] p. 3 (June).

Mr. Bennett. steel; nor had they proved to be "rustless" in actual practice salt water.

Dr. Desch. Dr. C. H. Desch thought that the extensive results broug forward by the Authors served to emphasize the complexity of t process of corrosion. It was evident that the many factors whi were concerned interacted in a complex way, so that it was excee ingly difficult to draw conclusions which would hold good over a wi range of conditions. The results would have to be compared wi those that had been obtained by the Corrosion Committee of t Iron and Steel Institute, most of which, however, covered a short period. The present results were of especial interest on account the long period of exposure.

The totally-submerged specimens showed comparatively sma differences (the highly-alloyed steels being excluded), and under suc conditions it would seem that chemical composition exerted only small influence on general corrosion; much greater differenc were, however, found in the extent of pitting, in which textu played an important part. It was natural that ingot iron, a highl oxidized steel low in carbon, should differ in its resistance to loc attack from wrought iron, which was very little oxidized and ha the property of deflecting pits on account of the linear arrangeme of the cinder inclusions. It was rather surprising that the differenc in that respect was not even greater. Atmospheric and half-tic specimens naturally showed greater variations, both among simil specimens at the same station and between the same steels at differe stations. Under such conditions it would appear that protectio would have to be given by painting, and that inherent resistance t attack, short of using highly-alloyed steels, was not to be expecte although the advantages of adding copper to mild steel exposed onl to the atmosphere were confirmed.

The results of exposure in the Gulf of Paria were of particul interest, and it was to be hoped that the mechanism of pitting in th highly-alloyed steels would be further studied. The extent of th organic growths on those specimens was unusually great, and th different degree of adhesion of barnacles and similar organisms t nickel and chromium steels might deserve study. In view of th complexity of corrosion under marine conditions, it seemed desirab that an analysis of the results obtained by the several committe should be examined by strict statistical methods, provided that th number of check specimens were sufficient; the body of resul presented by the Authors ought to form most valuable materi for such an investigation.

Mr. Elliot. Mr. T. G. Elliot observed that it had been a great pleasure f him to have assisted in the analyses required in the research. The

s a point in the Paper that might usefully be stressed, namely, e evidence afforded against the still prevalent idea, referred to on 7, that each metal possessed an inherent "corrodibility." That operty was now proved to be dependent upon the particular cumstances in each case. It might also be useful to mention ain the distinction between general corrosion and pitting, and pecially to draw attention to the suggestions made on pp. 19 and , which might be found useful in the consideration of later parts the Paper. Mr. Elliot.

Dr. Ulick R. Evans observed that an authoritative review of the tensive results of the corrosion researches of The Institution was great value. The calculations of Tables IX and X, which showed arly that mill-scale led to pitting, although on the average it had appreciable influence on the corrosion as a whole, were particularly luable. The remaining Tables in which materials were compared ere also of great interest. In applying the data, however, it would ve to be remembered that the order of merit of materials, when ed in the painted state, might not be identical with the order of the me materials when tested in the unpainted state. Under atmoheric conditions, the two orders were not very different, as shown a recent $2\frac{1}{2}$-year test by Mr. K. G. Lewis and himself.[1] There was ason to think, however, that under marine conditions the two orders ould show greater divergences. If the paint were applied to a eathered surface, the relative behaviour of different types of steels as likely to be determined mainly by the state of the mill-scale ached after the normal period of weathering. Probably the ideal arine steel would be one which would lose its scale very quickly on eathering. The state of the mill-scale at the moment of painting was pt to be more important than the composition of the steel as such. nother series of tests,[2] in which specimens had been weathered r different periods before painting (including also specimens mpletely descaled, mechanically or chemically), indicated that, under tmospheric conditions, the worst results occurred where the scale ad been removed locally by the weathering, the main areas remaining vered with scale at the time of painting. Intense attack occurred the breaks in the scale, and the rust formed forced away scale and aint together. On plates completely descaled by weathering efore painting, the paint remained firm for long periods. That ccorded with the general electrochemical principles governing the ntensity of attack at breaks in a "cathodic coat" upon steel, Dr. Evans.

[1] K. G. Lewis and U. R. Evans, Iron and Steel Institute Corrosion Committee eport, vol. 3 (1935), p. 177.

[2] *Ibid.*, p. 173.

Dr. Evans. whether that coat were copper or mill-scale.[1] The strength of electric current flowing between the coating as cathode and exposed steel as anode was largely determined by the cathod reaction. If the cathode were a large one (that was to say, if coating were nearly continuous), the current would be strong. T attack would be concentrated on the bare steel (as anode) and the exposed areas were small, the intensity of attack (namely, t attack per unit area) would be high, leading to pitting. It should remembered that on high-chromium steel the invisible oxide-fi formed at ordinary temperatures would prevent attack over t greater part of the area; if attack should occur, it would be severe localized and therefore intense. That explained the pitting to whi the Authors referred. On ordinary steel, the invisible film was t porous to localize the attack under the same conditions, and localiz attack (pitting) only occurred if mill-scale were present.

The pitting at breaks in mill-scale was to be expected even if t oxygen-concentration were uniform. If the oxygen-concentratio at the break were lower than that over the scale, there might be slight increase in the current-strength, but such differences in oxyge concentration were not needed. He did not remember that he ha ever used the differential-aeration principle to explain pitting interruptions in a mill-scale coat, as the Authors seemed to imply.

The differential-aeration principle did not seek to provide a explanation of all cases or types of corrosion, but it did explain th distribution of attack in salt solutions under conditions where oxyge had better access to some parts than to others. The distributio certainly seemed to call for explanation, since, although oxyge stimulated corrosion, the part nearest to the source of oxygen wa frequently seen to be immune. That was explained, not onl qualitatively but even quantitatively,[2] by the differential-aeratio currents which flowed between the relatively aerated part as cathod and the relatively unaerated part as anode, producing attack on the latter. But, as he had frequently pointed out in various Papers quite different distributions, involving corrosion at the water-line were also met with, and, furthermore, corrosion-currents could be set up by factors other than differences in oxygen-concentration.

Dr. Footner. Dr. H. B. Footner observed that there were definite indications in the Paper that rolling-mill scale had been the cause of serious pitting of steel plates, but that there was very little difference between the general wastage of specimens originally carrying mill-

[1] U. R. Evans, "The Corrosion of Metals at Joints and Crevices." Journa Roy. Soc. Arts, vol. LXXV (1926–27), p. 544.

[2] U. R. Evans and T. P. Hoar, "The Velocity of Corrosion from the Electro-chemical Standpoint." Proc. Roy. Soc. (Series A), vol. 137 (1932), p. 343.

ale and of others which were previously descaled. The fact that Dr. Footner. neral wastage had been similar, and, if anything, less on the plates om which the scale had not been removed, did not seem material, nce it was to be expected that, under the conditions of exposure, protected mild-steel plates would have a marked tendency to st. What was clearly brought out, however, was that the presence mill-scale led to marked pitting. That indication was fully borne it in practice, and an illuminating instance of the effect might be oted. At a petroleum installation in the Near East the contents different tanks from a certain date consisted of a semi-refined oil ving an appreciable organic acidity. In all the tanks a salt-water ottom was carried. After from 18 months to 2 years it was found at the bottoms of some tanks were actually holed, whilst others owed no pitting at all. Those tanks in which the bottoms were not tted had been in other service for 5 or 6 years, so that the mill-ale had been gradually and completely removed, whereas those in hich the bottoms were pitted were new and carried mill-scale when ey were first filled with the semi-refined oil, other conditions being entical in all the tanks.

Apart from its tendency to cause pitting, the presence of mill-scale ad a most pernicious effect on such structures as petroleum tanks gas-holders, which had to be painted. If paint were applied to ates carrying scale the protection obtained was almost invariably ad. That was due to two causes, the action between the oxide ale and the metal, which led to rusting, and the poor adhesion of e paint on surfaces covered with scale.

The latter point had generally been overlooked, but he had seen nk plates, some years after painting, on which the paint adhered ell and was quite elastic where the steel was free from scale but was rittle and lifeless on the surfaces where mill-scale still remained. here was a certain amount of rusting on the surfaces carrying scale, ut even where rusting had not yet begun the paint had definitely erished. In that respect a surface carrying scale resembled a ewly-galvanized plate which, as was well known, gave poor adhesion paint, the effect being noticeable under the action of sunlight.

It was now generally recognized that steel should be freed from ill-scale before being painted, and various methods of descaling ere available. One method was to expose the steel to the effects f the weather, but there were several objections to that simple ractice. It was often undesirable, for commercial or æsthetic easons, to leave structures unpainted for an indefinite period which ight, in some climates, be as long as 2 or 3 years. Furthermore, fter weathering, plates were covered with rust and dirt, as well as alt deposits in a maritime location, and very careful scraping was

Dr. Footner. necessary before the application of paint. It was a common fallac[y] that wire-brushing was sufficient after weathering, but cleaning [in] that manner certainly did not sufficiently free the plates from ru[st] or other deposits. Another method of descaling was by sand[-]blasting, which certainly provided an excellent surface for paintin[g]. It was necessary, however, to paint very soon after blasting, as th[e] exposed surface was very liable to corrosion. In addition, the proce[ss] required somewhat costly equipment and a supply of suitable san[d]. Another objection was its effect on the health of workmen, as it wa[s] liable to produce silicosis after a comparatively short period.

Descaling could also be effected by acid pickling. Until recen[t] years that process had been open to the objection that traces of acid or salts were left on the surface of the steel, which were liable to caus[e] corrosion and breakdown of a paint film, but methods of acid picklin[g] had now been devised and adopted in the petroleum and othe[r] industries which were free from that objection, and were at the sam[e] time entirely economical in practice. Mr. J. P. Pheiffer had described[1] a method of pickling steel plates or tubes, which consisted, briefly, o[f] immersing the steel in a hot 10–15-per-cent. solution of phosphori[c] acid and then in a dilute bath containing in solution about 2 per cen[t]. of free phosphoric acid and 0·5 per cent. of iron, and maintained a[t] 85° C. The first bath effected the descaling, whilst the second bat[h] functioned as a wash bath and at the same time deposited on the stee[l] surface a very thin film of ferric phosphate, which had a definit[e] inhibitive effect against corrosion in the atmosphere. That metho[d] of pickling had been adopted with success by certain petroleu[m] companies, but its cost was somewhat high. The process wa[s] cheapened by a discovery, covered by a French provisional patent[,] that the phosphoric acid in the descaling bath could be regenerate[d] by sulphuric acid with no deleterious effect on the pickled plate[.] Further investigation in Great Britain had shown that completel[y] satisfactory results were obtainable by descaling the steel in [a] 5-per-cent. solution of sulphuric acid, followed by washing in ho[t] water (65° C.) and final immersion in the hot dilute phosphoric aci[d] bath described above. Actual practice on steel tank plates had show[n] that that method of pickling completely removed mill-scale and lef[t] the steel in an ideal condition for painting. Plates pickled in that wa[y] showed no more tendency to corrode than when the scale was remove[d] by phosphoric acid alone. The process was economically sound, an[d] cost considerably less per square foot of surface than sand-blasting.

With any of the above pickling processes it was good practice t[o] paint immediately after pickling and, if possible, whilst the plat[e]

[1] "Pickling Wrought Iron and Steel by means of Phosphoric Acid." Worl[d] Petroleum Congress Proceedings, London, 1933. Vol. I, p. 542.

was still hot, since the paint penetrated better and was more easily worked into the surface of the plate. Actual practice during 2 years had shown that tank plates which were pickled and immediately painted with red-lead primer could be transported abroad and erected with no more than 5 per cent. damage to the paint film. That surprising result was no doubt due to the good adhesion of the film, since on corresponding plates which had been shop-painted over the scale the paint was invariably damaged during transport and erection. Dr. Footner.

The methods of pickling mentioned above had been given a thorough trial in the petroleum industry, and had been the means of preventing pitting of tank bottoms and roofs; they would result in a very considerable saving of repainting costs, since scraping to bare metal would no longer be necessary when repainting, the only requirement being a periodical renewal of the finishing coat. The process had recently been extended to the shipbuilding industry, the complete hull of a 7,000-ton ship having been pickled and shop-painted at a yard in Amsterdam. The red-lead paint applied to the hull after pickling had remarkable tenacity, and it was probable that the usual hull corrosion found on a new ship would be completely eliminated.

The most important points to be noted about the methods of pickling described above were that the steel surface after pickling was in an ideal condition for painting, and that there was nothing left on the surface which could cause corrosion or deterioration of the paint film. The thin film of iron phosphate was not resistant to corrosion by water, but had some inhibitive effect on atmospheric corrosion. Plates pickled by those methods, when left unpainted and exposed to the weather, showed even rusting after some time but no tendency at all to pitting. It should be mentioned that the process of pickling in sulphuric acid, followed by washing in water and immersion in the dilute phosphoric acid bath, was entirely free and was not covered by patent.

Dr. J. Newton Friend wished to call attention to the very great services which Sir Robert Hadfield had rendered to the Sea-Action Committee in its various activities. In the course of preparing and examining the metals to be exposed by the Committee at various stations, Dr. Friend had had occasion to spend many days at the works of Messrs. Hadfield's Ltd., in Sheffield, and he was glad to have the opportunity of publicly thanking Sir Robert and his staff, more particularly Mr. S. A. Main (one of the present Authors) and Mr. T. G. Elliot, for their unfailing kindness, courtesy and assistance. Without their generous co-operation the work of the Committee could not have been so efficiently carried out. The close concordance which had been found between the results at different stations and over different periods of exposure was in a large measure due to the Dr. Newton Friend.

Dr. Newton Friend.

great care with which the iron and steel bars were manufactured Sheffield; had the specimens not been prepared with the great skill it was obvious that the results obtained could not have be satisfactory, even if the later experimental technique had been perfe

The researches of the Committee had already extended over period of nearly 20 years, and during that time enormous progre had been made by various associations in the study of corrosion the great value of the data before them lay in the fact that they we not the results of short-period exposures of merely a few hours, day or even weeks, which might give very conflicting results. The da under consideration were the product of 5 and 10 years of testin and were therefore worthy of most careful consideration.

He would like to support Dr. Vernon's plea for the avoidance the possibly ambiguous use of the term "scaled" to indicate ste from which the scale had been removed. Dr. Vernon had also me tioned, giving reasons therefor, that if the Committee's bars had bee exposed inland very different results might have accrued. A simil idea had occurred to Dr. Friend, and upon his request in 1921 S Robert had generously placed at his disposal bars of ten varieties iron and steel, nine of which were specimens left over from th Committee's research. The bars measured 12 inches in length an had otherwise the same dimensions as the Committee's bars, and wer exposed in the "blue" condition, namely, with mill-scale sti adherent, except at the ends, which were machined and stampe for identification. The bars were placed in a rectangular woode frame, their ends being embedded in a thick layer of putty let int grooves in the frame. (Concrete, which had been found to be a excellent embedding material in the Committee's experiment could not well be used with a wooden frame.) The frame was lai horizontally on a flat portion of the roof of the Technical Colleg Birmingham in May, 1921. At intervals it was turned over to giv both sides comparable treatment. It should be remarked that th College was situated unpleasantly near to the L.M.S. railway statio and goods yards, so that it received no small quantity of smoke an soot from the engines. In May, 1927, after an exposure of 6 year the frame was dismantled; the wood had rotted seriously and th frame readily fell to pieces, but the hardened putty adhered to th ends of the bars like cement and had to be chipped off. The met protected by the putty was bright where it had been originall machined, and the identification lettering was clear and free from rus

After careful cleaning the losses in weight of the bars were dete mined, using the same balance and weights as those employed in th Committee's research, so that from a manipulative point of view th results were strictly comparable. The results were given in Tab

Dr. Newton Friend.

XXXIII of Sir Robert's Paper (p. 96). It was, however, instructive to compare the relative corrodibilities of the metals with those found by exposure at three of the Committee's stations. Table XXXIX gave that comparison, the Colombo station being omitted, as the aerial tests, owing to excessive spraying from the sea, closely resembled the half-tide tests. As no wrought irons were exposed it was convenient to choose the mild steel bar E as standard, its loss in weight being taken as 100, the losses of the others being expressed relatively thereto.

TABLE XXXIX.

Bar.	Type of steel.	Relative corrosion.			
		Birmingham, 6 years.	Halifax, 5 years.	Auckland, 5 years.	Plymouth, 5 years.
E	Mild steel, low S and P.	100	100	100	100
B	Mild steel, low Mn; high S and P.	161	163	152	136
F	Mild steel, 0·7 per cent. Mn.	114	122	87	124
D	0·40 per cent. C steel.	122	98	118	122
G	Mild steel, 0·6 per cent. Cu.	66	63	46	78
H	Mild steel, 2·2 per cent. Cu.	70	46	30	44
K	3·75 per cent. Ni steel.	41	27	26	17
L	36·55 per cent. Ni steel.	6	2·7	3·5	1·4
J	13·6 per cent. Cr steel.	1·5	11	9	2·6

Consideration of the Table showed that, except for the steel H, the results were reasonably concordant; although bar L had lost more than bar J in the Birmingham tests, the losses of both metals were small and a few grams made all the difference.

Had the data been available it would have been interesting to compare the behaviour of those metals towards typical country air such as that of the Sussex Downs, and moist air such as that of Borrowdale, Cumberland.

In the aerial tests pitting was not usually serious and the cast irons did not suffer appreciable graphitization; hence the losses in weight might be regarded as fairly accurate measures of the extents of corrosion.

In the half-tide and immersion tests, however, both pitting and, in the case of the cast irons, graphitization were appreciable and rendered the problem more complex. The cast irons proved very disappointing in their resistance to sea water; they underwent serious graphitization, and were often internally corroded, so that their external appearance was deceptive, giving little or no indication of their weakness. The experimental results were in complete accord

Dr. Newton Friend.

with the observations recorded[1] of the corrosion of the landin stage of the Birnbeck Pier at Weston-super-Mare. The introductio of alloying elements such as copper, chromium and nickel might ten to reduce the losses in weight of the steels exposed to sea-action but in the Committee's experiments there appeared to be an increase tendency for those steels to undergo localized corrosion or pitting That result was evidently supported by the observations of th Authors on the specimens immersed in the Gulf of Paria. Th 36-per cent. nickel steel and the 18–8 chromium-nickel steel would of course, be extremely expensive, and their superiority over ordinar steels and low alloy steels would have to be overwhelmingly grea for them to receive more attention.

Dr. Hatfield.

Dr. W. H. Hatfield, Chairman of the Joint Corrosion Committee o the Iron and Steel Institute and the British Iron and Steel Federation observed that the analysis which the Authors had prepared of th results of the Sea-Action Committee's tests on unpainted bars should be of considerable utility as a general survey of those tests. Even with a limited series of irons and steels, a very large number of test would be required to cover fully all the possible variations in con ditions of exposure and pre-treatment of the materials which wer of practical interest, and there were directions in which it would have been of considerable interest to extend the work under con sideration. Nevertheless, the tests had produced a number of very interesting observations which should not only assist those responsible for the choice of materials for marine structures, but also stimulate interest in further tests.

The long time usually required for the completion of corrosion tests was particularly unfortunate where tests on the special alloy steels were included because in that rapidly developing field the steels tested were liable to be superseded by improved types long before the final results became available. The research under discussion had, in fact, suffered in that way, notably in the case of the 13-per-cent. chromium steels, which for marine work had now been almost entirely superseded by the higher chromium materials, usually of the 18/8 chromium-nickel type.

Another difficulty encountered in such tests was in the design and carrying out of sufficient tests to show clearly the influence of any particular factor. Taking, for example, the present tests with carbon steels, the results suggested that under aerial conditions the steel low in manganese and high in sulphur and phosphorus was inferior to the other materials, whereas under immersed conditions it was the least corroded. Before it would be possible to say whether or not

[1] Fourth (Interim) Report of the Committee on the Deterioration o Structures exposed to Sea-Action, p. 28. London, 1924.

those results were in any way related to, say, the sulphur and phosphorus content, it would be necessary to carry out a series of tests to explore various other possibilities. For instance, the increased corrosion might well be connected with the condition of the oxide scale resulting from the particular rolling treatment given during production, and might have little or no relationship to the composition of the steel. It would be appreciated, therefore, how difficult and complicated a matter it was to obtain specific information as to the influence of the various factors, and why it was that the Corrosion Committee of the Iron and Steel Institute and the British Iron and Steel Federation had already found it necessary to expose nearly two thousand specimens and was actively engaged in the preparation and exposure of some hundreds of further specimens, in addition to the planning of new series of tests, whilst, as yet, its work had scarcely touched the higher alloy steels. Again, it was very desirable that tests such as those under consideration should be supplemented as far as possible by a study of the behaviour of steels under actual service conditions. That was a necessary phase of the work, since it was often impossible to allow for all the various factors which might operate in service. Dr. Hatfield.

Considering the detailed results, the indicated advantage resulting from the addition of 0·6 per cent. of copper to the carbon steel continuously immersed in sea-water was decidedly interesting, especially in view of the general indication from other tests that small additions of copper were not beneficial under such conditions. In that connection it would have been useful to have included a similar steel with 0·2 to 0·3 per cent. of copper, which would have permitted the direct checking of the Authors' suggestion that, whilst 0·2 to 0·3 per cent. of copper was fully effective in industrial atmospheres, a larger amount was required before the influence became apparent in sea-water. In the circumstances, it would be particularly interesting to study the comparative behaviour of strictly comparable 0·2-per-cent. copper steel and 0·5-per-cent. copper steel specimens which the Corrosion Committee of the Iron and Steel Institute had arranged to expose to marine conditions very shortly.

In the tests which the Authors had carried out in the Gulf of Paria, the conditions of exposure had certainly been very drastic and had evidently been such as to hinder seriously that reasonably free access of oxygen to the surface of the various austenitic chromium and chromium-nickel steels which was necessary if they were to maintain their full corrosion-resistance in media such as sea-water. It had been amply demonstrated that the 18/8 chromium-nickel type of steel behaved excellently in marine service where oxygen-shielding was not a serious factor; that applied even under static conditions

Dr. Hatfield.

of exposure, as, for example, in the case of the rods driven into th Reef as survey-stakes by the Great Barrier Reef Expedition · 1928–9. The Authors implied that the relatively small attac obtained with the pickled sample of the 10-per-cent. chromium 20-per-cent. nickel steel No. 3765 in the Gulf of Paria tests migh have been due to its lower chromium and higher nickel content i comparison with the other steels in the group. That seemed rathe improbable, particularly in view of the much higher loss shown b the 36-per-cent. nickel steel. It would seem much more probabl that the sample in question had a particularly passive surface, sinc considerable variability in the passivity of pickled samples of suc material could occur. Undoubtedly, the most generally usef special steel for marine work was the well-known 18/8 chromium nickel type.

Drs. Honda and Endo.

Drs. Kotaro Honda and Hikoto Endo, of Tôhoku University observed that, although many metallic and environmental factor were involved in the corrosion of iron and steel in salt solutions the possible actions of each factor had now been sufficiently clarifie by various investigations. In laboratory experiments those factors were comparatively limited, but in the more complex conditions o exposure-tests important factors might be revealed. The results obtained from such tests often contradicted conceptions forme from laboratory work; that was undoubtedly due to the predomi nance of those factors which acted only under marine exposure Hence, also, it was generally true that the same results would no be obtained from the same material if there were differences in the exposure conditions, location, character of sea-water at differen seasons, and so on. The amount and character of the corrosion observed were the results of a combination of factors, each of which might be different in different locations or under different circum stances. It was, therefore, important and significant that authentic results should be accumulated from practical exposure-tests made in various locations, one of the major objects of such tests being to find out the predominating factors affecting corrosion and thei mutual accelerations and retardations at each location.

The results of the Author's extensive investigation would be very instructive to engineers; to laboratory men they suggested many important problems to be studied, especially with regard to pitting According to the Authors' results, the amount of pitting did not seem to follow any systematic relationship with respect to locations o methods of exposure, but seemed to be affected by many factors and to be caused by complicated reactions. For example :—

(i) At Halifax, the extent of pitting was extraordinary under half tide conditions, in spite of the fact that it was very slight under tota

immersion and aerial conditions. The character of the sea-water and the molluscs at Halifax apparently contributed some special corrosive feature causing that excessive pitting under half-tide conditions only. The contamination of water with oil at that port might perhaps be responsible. It was therefore desirable that the mechanism of the reaction of oil-contaminated sea-water on iron and steel—for example, whether a colloidal action of oil was an accelerating factor or whether an oil layer acted as a protecting film—under half-tide conditions, should be investigated. Drs. Honda and Endo.

(ii) At Colombo, the pitting on a steel surface was greater under half-tide than under aerial or total-immersion conditions, whereas for iron it was less than in the latter two cases. That great difference in results between iron and steel might be attributed not merely to the difference of carbon content but perhaps also to factors of environment. Perhaps the Authors would be able to give an explanation.

(iii) In general, the rusting of iron and steel in air in winter, when the mean relative humidity was comparatively high, was greater than that in summer, when the mean relative humidity was small; also, the velocity of rusting of iron on which rust already existed accelerated greatly over the critical humidity-range between 60 and 70 per cent., as confirmed by Dr. Vernon. On the contrary, however, the rusting in air at Colombo, in a hot climate, was greater than that at Halifax, in a cold climate. In that case the relative humidity was not the major factor, as the surface of a specimen near the sea was maintained in a wet condition by salt spray, but the difference of air-temperature between the two places was an important factor, as an increased temperature accelerated the reaction-velocity. The nature of the sea-water, and other factors, had also to be taken into consideration.

(iv) It was a widely accepted view that the corrosion-velocity of iron and steel in alternately wet and dry conditions was larger than that in total-immersion or aerial conditions. In the tests described in the Paper, however, the loss in weight of the large majority of ordinary steels and rolled irons under the half-tide conditions was intermediate between the losses experienced under aerial and total-immersion conditions. The above result might be due to the salt-spray, which maintained the surfaces of the specimens in a wet condition, and also to the character of the sea-water at the locations where the experiments were made. That view, however, did not apply to the results of the half-tide tests on rolled irons and steels at Auckland and Plymouth, where the loss in weight was usually less than that under either aerial exposure or total immersion. The

Drs. Honda and Endo.

causes of that difference might need further experiments for th elucidation.

Although there were many at present unaccountable results, su as those mentioned above, Drs. Honda and Endo were gratified find that most of the results could be explained by their own findin on the following principles :—

(1) In total-immersion conditions :—

(*a*) Additions of small amounts of special elements to iron an steel had a very small effect, which could safely be neglected comparison with other factors in neutral salt-solutions. T corrosion of iron specimens in those conditions was affected chief by the metallic factors (i) surface condition, (ii) presence of differe phases, (iii) design (form) of structures, (iv) protective film; an by the environment factors (v) concentration of oxygen, (vi) co centration of hydrogen ions, (vii) adhesion and distribution corrosion-products, (viii) electrical conductivity of solutions, etc. A small addition of special elements had a large effect on th dissolution of iron by acid in total-immersion conditions, and al in aerial and wet-and-dry conditions, but it had almost no effe under total immersion in neutral salt-solutions, and the total wasta of ordinary iron and steel was almost the same if the concentratio of chlorine ions and of oxygen and acidity in solutions were almo equal, as was seen in *Figs. 4* (p. 25). However, it was natural tha when the content of special elements was increased to more than 7 o 8 per cent., as in 13-per-cent. chromium steel, 18/8 chromium-nick steel, or 36-per-cent. nickel steel, the added element had a considerab effect even in the neutral solution under total-immersion condition

(*b*) It had been ascertained by their own experiments that the lo in weight of iron and steel in water and sodium chloride solution wa greatest at temperatures between 60° and 70° C. The effect o climatic temperature was very small in that experiment, as th temperature-difference for all locations from the hottest to th coldest climates was not so large under total-immersion condition The character of sea-water in that case seemed to be a mo important factor than the temperature, but in aerial and half-tid conditions the effect of temperature was clear, indicating that it wa a predominating factor; for example, the loss in weight in aeri conditions at Colombo was greater than that in the colder climate o Halifax.

(*c*) It was also well known that if the form of specimen were ve complicated, the velocities of corrosion at various parts would b different owing to the difference of oxygen supply, and hence th results obtained would not be the same as those found from a specim

of simple form. Another important factor was the position of specimens with respect to the current, as changes of position might affect the adherence of corrosion-products to the surface of the specimen. Drs. Honda and Endo.

(d) The chemical character of sea-water at a location had a predominating influence on the corrosion of iron and steel; it might be affected by substances discharged from ships and also by molluscs or other fauna which adhered to steel structures. If the composition and seasonal change of sea-water were more thoroughly investigated, the curious results might be effectively explained. It was interesting to find that the character of sea-water at Halifax was such as to cause greater corrosion of iron than that occurring at other ports.

(2) In aerial conditions:—

(a) As the chemical composition and adherence of corrosion-products depended upon the amounts and properties of added elements, a small amount of special element added might have an appreciable effect on the rusting of iron and steel in air. In the case of copper-bearing steel, for example, the small content of copper formed a protective layer of copper oxide during the progress of rusting. The effects of such small amounts of added elements and impurities were evident in *Fig. 11*, p. 59.

(b) The mill-scale of iron and steel had a protective power against rusting and corrosion, and the total wastage of scale-covered iron was smaller than that of de-scaled iron. If there were fissures or rents in the scale, pitting would easily occur on the defective portions. The character of the scale varied with the amount of the added element and also with the heat-treatment employed.

(c) As the mean relative humidity in winter was greater than that in summer, the rusting of iron and steel was greater in winter than in summer. That experimental result appeared to be contrary to the fact that the rusting at hot Colombo was found by the Authors to be greater than that at cold Halifax; the difference, however, was probably due to the temperature-effect, which was the major factor in deciding the reaction-velocity between iron and sea-water.

(d) It was reasonable to consider that the content of sulphur dioxide (produced by the combustion of coal) in the air acted as an important factor in aerial corrosion.

(3) In wet-and-dry or half-tide conditions:—

(a) Small amounts of added elements had also an influence on the corrosion.

(b) They had often found in their own experiments that alternately wet and dry conditions were more favourable to corrosion than total-immersion or aerial conditions. The wastages under half-tide conditions obtained by the Authors, except in the case of cast iron

Drs. Honda and Endo.

at Colombo, were intermediate between those experienced und aerial and total-immersion conditions. The results could be part explained by the consideration that under aerial conditions Colombo the rusting was very large owing to the high temperatur and that under total-immersion conditions at Halifax the wastag was also comparatively large owing to the character of the sea-wate The corrosion under half-tide conditions of the rolled iron and ste at Plymouth and Auckland, however, was generally less than tha under either aerial exposure or total immersion, and at Auckland was particularly small. That difference could not be clear explained.

(4) Pitting :—

(*a*) Pitting was generally produced by the formation or th previous existence of fissures and rents in the protective films, suc as mill-scale or the very thin oxide film which might be found o stainless steel. Accordingly, the nature of mill-scale, and th influence of added elements and heat-treatment on its chemical an physical properties, was worth a thorough study. It was we known that the number and the depth of pits produced on the surfac of a stainless steel changed with the relative contents of chromiur and carbon and with the tempering temperature. Likewise, th corrosion-resisting properties of all kinds of protective films on variou ferrous alloys required further study in the laboratory.

(*b*) It was readily understandable that complete immersion i sea-water definitely favoured pitting as compared with aerial o half-tide conditions, because sea-water was an efficient electrolyte When anodic areas were very small as compared with cathodic area on the surface of a specimen, the depth of pitting became large a the current-density increased. The direct contact of iron wit sea-water for a long time was therefore a necessary condition for th occurrence of severe pitting. The moisture which collected on a specimen exposed to the air, or small amounts of sea-water brought o the surface by salt-spray, were not so effective.

(*c*) In general, the total wastage of specimens due to mill-scale o very thin oxide film was very small, notwithstanding their tendenc to induce pitting. When the total wastage was small, the pittin was deep, and when the total wastage was large and pitting occurre at the same time, the pitting was generally very shallow. Most o the results obtained by the Authors could be explained by the abov conceptions. The fact that the depth of pitting at Auckland wa shallow, in spite of the small total wastage, might be explained b the small ratio between the anodic areas and cathodic areas and als by the electrical conductivity of sea-water at Auckland. The dept

of pitting under half-tide conditions at Auckland, however, was less than under total-immersion or aerial conditions; the explanation of that fact seemed to require a further investigation. Drs. Honda and Endo.

(*d*) The temperature had also an influence on the occurrence of pitting, which was severer in a hotter than in a colder climate. In fact, the pitting of steels increased, on the average, in the order of Halifax, Plymouth, Auckland, and Colombo. Irons, however, suffered less pitting at Colombo than at Auckland; the cause of that result was not clear.

(5) Special steels :—

According to experiments made by Drs. Honda and Endo, copper had no effect on the corrosion of iron in neutral salt solutions under total immersion, but when a small amount of aluminium was added to copper-bearing steel, it became more resistant against corrosion in neutral solutions. The addition of 0·35 per cent. of copper (which was the solubility-limit of copper in α-iron at room temperature) increased the resistance to dissolution in hydrochloric acid of various concentrations and sulphuric acid of various concentrations below 50 per cent. The experiments and their explanation would shortly be published by Dr. Endo.

Their experiments had also confirmed that 36-per-cent. nickel steel was very resistant against corrosion in neutral salt-solutions, and had almost the same resistivity as pure nickel against dissolution in 10-per-cent. sulphuric acid. Although the 36-per-cent. nickel steel was one of the most resistant alloys, it did not escape corrosion by sea-water. The 18/8 chromium-nickel steel and 36-per-cent. nickel steel were easily attacked by solutions of mercuric chloride, ammonium chloride, hydrochloric acid, and sulphuric acid. The alloy which they considered the most resistant consisted of 15 to 20 per cent. chromium, 7 to 10 per cent. molybdenum, 10 to 20 per cent. iron, 1 per cent. manganese, and the rest nickel, but as it was more expensive than those mentioned just above, it was not suitable for use in large structures. As a cheaper alloy, they adopted a steel containing aluminium, chromium, silicon, and copper, of which the total addition was below 8 per cent., as in "Chromador"; under total-immersion conditions that alloy was more effective than "Chromador."

Mr. JAMES KEWLEY observed that the attention paid to corrosion problems at the present time was reflected in an exhibition on corrosion which had been arranged by the Asiatic Petroleum Company, and had been held in London. Although one of a series of exhibitions primarily intended for the education of the staff of that Company, it had afforded a very valuable opportunity for the discussion of Mr. Kewley.

Mr. Kewley. corrosion problems of vital importance to the petroleum indus[try] with some of the authorities on corrosion, and with suppliers of equ[ip]ment for oil-refineries.

The exhibition dealt with the factors affecting corrosion and w[ith] means adopted to avoid or reduce corrosion in air, water, and s[oil] and under refinery conditions. Particular attention was paid to t[he] selection of the proper grade of paint and to the necessity of treati[ng] steel plates before painting. A section of the exhibition dealt w[ith] the results of galvanic action between dissimilar metals, which w[as] strikingly illustrated by the use of the ferroxyl indicator. Ot[her] features included were the methods of protection of pipe-lines ([by] coatings and electrical methods) the detection of pinholes in coatin[gs] and the effects of anaerobic bacteria on steel. A collection of spe[ci]mens of corroded refinery equipment emphasized the particul[ar] problems confronting the oil industry.

Although the exhibition had now been closed and most of t[he] exhibits returned to their owners, a complete catalogue was bei[ng] prepared, and might be found of assistance to any research associ[a]tion or institution considering an exhibition along similar line[s]. The Asiatic Petroleum Company would be very glad to offer an[y] assistance in that direction that might be required.

Dr. von Wolzogen Kühr. Dr. C. A. H. VON WOLZOGEN KÜHR, of Bloemendaal, Hollan[d], observed that, besides the ordinary or aerobic corrosion of iron (wit[h] oxygen from the air) causing the formation of yellowish-brown rus[t], the anaerobic corrosion of iron (without oxygen from the air) in th[e] soil was of great significance. With the first-mentioned process th[e] iron in the product of corrosion was characterized by the ferric stat[e], but with the second by the ferrous state.

The anaerobic corrosion of iron had been studied in the Netherlan[ds] since 1922, and was found to be principally of a micro-biologic[al] nature. The primary iron corrosion process :

$$Fe + 2H^{\cdot} \rightarrow Fe^{\cdot\cdot} + 2H$$

continued uninterruptedly only when a depolarizer (hydroge[n] acceptor) was present. Suitable depolarizers were found to be :—

(i) The sulphate reduction of Beijerinck (1895), according t[o] the equation by Baars :

$$H_2SO_4 + 8H \rightarrow H_2S + 4H_2O$$

The reaction of the iron corrosion was :

$$4Fe + H_2SO_4 + 2H_2O \rightarrow 3Fe(OH)_2 + FeS.$$

The curious fact that that frequently very severe corrosio[n] process took place in an approximately neutral mediu[m] (pH value = about 7) was explained by the formatio[n] of FeS and $Fe(OH)_2$, which had a neutral nature.

(ii) The carbon dioxide reduction of Söhngen (1906): Dr. von Wolzogen Kühr.

$$8H + CO_2 \rightarrow CH_4 + 2H_2O.$$

The corroded iron forming in that case passed into $Fe(OH)_2$, so that the pH value of the medium again remained about 7.

In the case of sulphate reduction the bound sulphate oxygen was the hydrogen acceptor, and in the case of the carbon dioxide reduction the oxygen from the carbon dioxide was the hydrogen acceptor. The sulphate reduction occurred quite generally everywhere, whilst the carbon dioxide reduction occurred in boggy and marshy districts. The two processes might also occur simultaneously. The anaerobic corrosion process was entirely the same for steel and for cast-iron pipes. In the case of steel pipes it resulted in pitting, whilst cast-iron pipes also showed graphitization owing to their higher carbon content (about 3·5 per cent.).

The anaerobic iron corrosions referred to were to be considered as electro-biochemical processes. Since, therefore, with both the aerobic and the anaerobic corrosion of iron pipes in the soil the attack of the iron had to be ascribed to the anodic solution of the iron by means of a depolarizer (hydrogen-acceptor), there existed, from an electro-chemical point of view, no fundamental difference between the two corrosion processes just mentioned. The unity manifested itself by the invariably present and essential hydrogen-acceptor. It was always possible, in cases of iron corrosion in the soil, to explain conformities and nonconformities by the prevailing conditions under which the iron corrosion took place. The integral parts of the mechanism of the anaerobic corrosion of iron were not by any means novel. On the other hand, the bond uniting the whole gave a novel insight into the process.

Mr. F. J. Maurice observed that under all the various test conditions discussed in the Paper, the ingot iron exhibited a resistance to corrosion appreciably lower than that normally shown within his experience and generally demonstrated under service conditions. The explanation of that difference, however, was apparent from the range of analyses of ingot iron given in Table IV, Appendix II. At the time when the test-pieces were procured, British-made ingot iron was in its infancy, although it had regularly been made in the United States during the previous decade, and was now in regular production both in Britain and in all the leading Continental steel countries. The analyses given in the Table cited showed a total of the five major impurities ranging between 0·168 and 0·202 per cent., Mr. Maurice.

Mr. Maurice.

but the current typical analysis of the metal as produced to-day Britain was :—

Carbon	0·015 per cent.
Silicon	0·005 ,,
Sulphur	0·035 ,,
Phosphorus	0·005 ,,
Manganese	0·020 ,,

the total being only 0·08 per cent. It was understood that British ingot iron was now manufactured to a guarantee of 0·10 per cent of the elements listed above.

Professor Portevin.

Professor A. M. Portevin, of Paris, observed that the Paper by Sir Robert Hadfield and Mr. S. A. Main constituted one of the most valuable contributions which had been made to the important problem of the corrosion of steel. Having himself, in collaboration with Mr. E. Herzog, studied that problem for several years at the steel works of Pompey, he presented the following notes and ideas with a view to comparing some of the results with those given in the Paper.

It had been found necessary to systematize and regularize the results of the various corrosion-tests carried out in the laboratory at Pompey by defining and laying down the working conditions and by introducing standards of comparison based on a Martin-type steel.

The comparisons which he proposed to make dealt with the following points :—

The rapidity of the average attack.
The distribution or type of attack.
The effect of the state of the surface and of scale.
The effect of the temperature.
The effect of the composition of steel, for :—

Nickel steels.
Copper steels.
Chromium steels.
Ordinary steels and rolled irons.
Steels with the addition of other elements.

The Rapidity of the Average Attack.

The rapidity of attack obtained with the three methods of testing used (salt mist, intermittent immersion, and total immersion) was as shown in Table XL, and compared with the results of the corresponding tests quoted by the Authors.

Professor Portevin.

TABLE XL.—AVERAGE LOSS IN THICKNESS: MILLIMETRES.

	Place of test.	Duration of test.	Saline mist or sea air.	Half-tide conditions or intermittent immersion.	Total Immersion. Still liquid.	Total Immersion. Agitated liquid.
Standard Martin steel [1]	Laboratory at Pompey	1 month	0·025	0·057	0·009	0·035
		2 months	0·044	0·109	0·016	0·065
		2½ ,,	0·081	0·154	0·024	—
		5 years (extrapolated).	1·38	3·00	0·480	1·8
Average ordinary steels and rolled irons	Halifax	5 years	0·140	0·388	0·513	
	Auckland	,,	0·300	0·186	0·430	
	Plymouth	,,	0·951	0·414	0·478	
	Colombo	,,	1·834	1·014	0·495	

[1] The standard Martin steel for the laboratory tests at Pompey (sheet from middle of ingot) had the following analysis :—

C	0·07 per cent.
Mn	0·41 ,,
Si	0·02 ,,
S	0·024 ,,
P	0·018 ,,

The laboratory tests gave, in increasing order of rapidity, the following classification :—

	Coefficient.	Loss in weight: grams per square metre per month at 30° C.
Total immersion in calm liquid .	1	60/70
Total immersion in agitated liquid	3–4	180/240
Salt mist	3	200
Intermittent immersion . . .	6	400/430

The practical results given by the Authors furnished a grading which was not the same; besides, it varied from one testing station to another. In general, the grading for increasing loss in weight was :—

(1) Salt air (either first or second, varying with the testing station).
(2) Half-tide conditions. (,, ,, ,, ,, ,, ,, ,, ,, ,,).
(3) Total immersion.

In the laboratory, corrosion was slowest under total immersion in still liquid. That was different from the results of tests in the sea, where currents and aeration probably intensified the attack. The figures derived for a 5-year period from those obtained in still liquid in the laboratory agreed with those obtained in the sea. In that

Professor Portevin.

test, therefore, there was no acceleration in the rate of corrosio It was difficult to work out how much more rapid the laborato tests were than the practical tests.

Taking the highest figures, which were those for Colombo, and th lowest, as observed at Auckland, the rates of corrosion in th laboratory tests relative to each of them were as given in the followin Table ; for still liquid the rates were found to be in agreement.

	Colombo.	Auckland
Salt air	0·7	4·5
Intermittent immersion	3·0	16·0
Total immersion (agitated liquid)	3·6	4·2
,, (still liquid)	1·0	1·12

The Distribution or Type of Attack.

In the laboratory tests on total immersion in still liquid and o alternate immersion and exposure, the ordinary Martin or Thoma steels showed a smooth surface hardly grooved at all. The attac was practically uniform. The test in salt mist exhibited fine pittin when the dimension of the droplets was of the order of 10 to 3 thousandths of a millimetre, provided that the spraying was inte mittent, followed by periods of drying. On the contrary, if the drop were larger, for example, several tenths of a millimetre, or if th spraying were continued without drying, the attack showed itse in fairly large cavities or grooves, the distribution of the corrosio becoming fairly uniform.[1]

The depth of the pitting obtained in fine mist was about 0·1 to 0·12 millimetre per month for the standard Martin steel. cupping test on a specimen showed a reduction of 15 per cent. in th cupping pressure ; as the loss of weight was only 4·2 per cent., tha reduction was to be attributed to the local corrosion.[2] The ductilit of the steel, as shown by the stretching of the surface in that tes was not diminished by 1 month's exposure to corrosion ; the pit although very fine, were rounded in form and their notch-effec was not appreciable.

[1] A. Portevin and E. Herzog, " Quelques conditions à réaliser dans les essa de corrosion des aciers en milieu humide," Comptes Rendus de l'Academie d Sciences, vol. 199 (1934), p. 789.

[2] E. Herzog and G. Chaudron, " Sur l'alteration des propriétés mécaniqu des tôles de duralumin après corrosion par l'eau de mer," Comptes Rendus d l'Academie des Sciences, vol. 189 (1929), p. 1087.

Professor Portevin.

In the tests in the sea, pitting was in fact general. That result differed completely from the results obtained by laboratory tests, and research into the cause of the difference would deserve most careful study. The average depth of the pitting in 5 years in salt air was from 0·8 to 1·2 millimetre, in half-tide conditions from 0·45 to 2·5 millimetres, and in total-immersion tests from 0·5 to 2·8 millimetres.

Condition of the Surface, and Influence of Scale.

In the intermittent-immersion tests and in salt mist, the standard Martin steel, when covered with scale, was attacked 20 to 30 per cent. less than the same steel with the surface polished with emery paper (No. 00 Denis-Poulot). The corrosion by pitting was, however, more pronounced. At the end of 1 month's test in the presence of scale the pitting was as deep as 1 millimetre. With a polished surface in salt mist, however, the pitting was only 0·1 millimetre deep, and it was not measurable in the case of the intermittent-immersion tests. That observation was in agreement with the results of practical tests, which showed that the depth of pitting was doubled in the presence of scale. It should be noted that the favourable effect of small additions (1 to 5 per cent.) of nickel, chromium, silicon, etc., disappeared if the samples tested were covered with scale. With polished surfaces the addition of those elements gave a diminution in the general attack varying from 20 to 30 per cent., in accordance with the composition and conditions of attack.

The scale—iron (magnetite—iron) couple had been the object of study in the laboratories of the Pompey steelworks, where Mr. Herzog had introduced a new technique for measuring the potentials and currents generated by the couples formed between iron and its compounds with sulphur, phosphorus, and oxygen. The difference of potential between magnetite and iron in salt water was 1 volt, the current being 0·002 ampere per square centimetre. The formation of pitting was explained by the large size of the cathodic surfaces in relation to the small anodic surfaces which existed on steel covered with cracked or discontinuous scale. The magnetite, which acted as the cathode, was formed by reaction between the yellow rust (Fe_2O_3, H_2O) and ferrous hydrate (FeO, H_2O). According to the results obtained by Mr. Herzog, it was a semi-conductor, its resistance in a wet state being from 3 to 4 ohm-millimetres. The rust (Fe_2O_3, H_2O) was an insulator, but it assisted the oxidation of hydrogen at the cathodes, acting as a depolarizing agent.

Professor Portevin.

Effect of Temperature.

Professor Portevin and Mr. Herzog had found [1] that for extra-so standard Martin steel in sea-water the temperature-coefficie between 20° and 40° C. was :—

(*a*) 0·20 to 0·30 in the tests of total immersion in calm liquid.
(*b*) 2·0 in the tests of alternate immersion and exposure. T considerable activity of the attack was due to the dryi of the wet flakes in the air.

The practical results given in the Paper were in excellent agreeme with their findings.

(*a*) In total immersion, corrosion differed little between th four testing stations, where the temperature varie considerably, from 28° to 33° C. at Colombo (Ceylon to 10° to 12° C. at Halifax (Nova Scotia). In fact, th depth of general corrosion ranged only from 0·43 to 0 millimetre between the two testing stations.
(*b*) In specimens exposed to half-tide conditions and to salt a the influence of the temperature was very marked. Th testing station at Colombo, situated in the hottest are showed the greatest attack.

The pronounced variations found in the tests with specime exposed to half-tide conditions and to salt air were probably als related to differences in the humidity, the rhythm of immersion an exposure, and the wind-velocity—factors of which the influence ha been shown in laboratory tests—and other well-known factors, suc as dusts and gases (sulphur dioxide, ammonia, or others pointed ou by Dr. Vernon) would also play an important role.

Effect of the Composition of the Steel.

(*a*) *Nickel steels.*—Tests made in the laboratories at Pompey ha given the following relative losses of weight :—

	Intermittent immersion.	Salt mist.
Standard Martin steel	100	100
2·3 per cent. Ni ; 0·08 per cent. C	50–65	75–70
5·6 ,, ,, ,, ,, ,, ,, ...	50	65–70
33·5 ,, ,, 0·17 per cent. C	4·4	3

[1] See footnote (1), p. 634.

Those results applied for the conditions described by Professor Portevin and Mr. Herzog in 1934 [1] for a temperature of 30° C. They were in agreement with previous studies carried out by Mr. Herzog in the laboratory of Mr. G. Chaudron at Lille, where the improvement in the intermittent-immersion tests was 40 per cent. for 2 per cent. nickel and 50 per cent. for 6 per cent. nickel. Professor Portevin.

The tests of the Sea-Action Committee at four ports gave the following relative losses :—

	Immersion.	Half-tide conditions.	Salt air.	Pitting. (all three conditions.)
Standard steel (0·40 per cent. C)	100	100	100	100
3·7 per cent. Ni., 0·2–0·3 per cent. C	75	63	54	77*
36 per cent. Ni	46	20	1·1	33

* Results from three ports only.

The improvement varied with the testing station and the method of testing. Thus, in the salt air at Colombo it was only 20 per cent., but at Plymouth it was 86 per cent. with 3½ per cent. of nickel. The favourable effect of nickel was especially notable in reducing the tendency to pitting. The comparison between the laboratory and practical tests was sufficiently satisfactory for steels with small percentages of nickel. Laboratory tests showed greater improvement than practical tests in the case of larger percentages of nickel, especially for intermittent immersion (half-tide conditions).

(*b*) *Copper steels.*—Laboratory tests did not show any advantage with 0·5 or 1 per cent. of copper. Sometimes even a 10 to 20 per cent. more rapid attack took place than for steels without, or with only 0·2 to 0·3 per cent. of, copper.

On the other hand, the tests of the Sea-Action Committee in certain cases showed appreciable decreases in the corrosion of steel through the addition of copper. Table XLI (p. 638) gave the relative losses compared with those of a standard steel containing 0·4 per cent. carbon, which were taken as 100.

In connection with the tendency to pitting, copper steels behaved as ordinary steels; sometimes they were even more pitted.

It would be seen from Table XLI that copper decreased the attack, especially in salt air, at all the testing stations with the exception of Colombo. At Auckland and Halifax copper steel still showed an advantage in the case of intermittent immersion (half-tide conditions). In total immersion the improvement from copper steel was not so marked. Summing up, it might be said that in salt air

[1] See footnote (1), p. 634.

Professor Portevin.

TABLE XLI.—RELATIVE GENERAL CORROSION OF STEELS WITH COPPER, EXPRESSED AS PERCENTAGES OF CORROSION OF CARBON STEEL.

	Total immersion.	Half-tide conditions.	Salt air.
(1) *Halifax.*			
D—standard with 0·4 per cent. carbon	100	100	100
G—soft steel with 0·5 per cent. copper	95	44·5	65
H—soft steel with 2 per cent. copper	97	50	47
(2) *Auckland.*			
D—	100	100	100
G—	84	61	39
H—	82	100	26
(3) *Plymouth.*			
D—	100	100	100
G—	87	98	64
H—	90	77	36
(4) *Colombo.*			
D—	100	100	100
G—	73	102	95
H—	87	114	83

the addition of 0·5 to 2 per cent. of copper gave better results tha for immersion in sea-water. With 0·2 to 0·3 per cent. of coppe there was no record of any decrease of corrosion.

All experimenters were in agreement that in sulphurou atmospheres a copper-content of 0·3 to 0·4 per cent. definitel decreased the attack. Atmospheric-corrosion tests carried out i the neighbourhood of the blast-furnaces of Pompey had in fact show 25 to 35 per cent. decrease in corrosion (after 1 year of exposure with the addition of 0·3 to 0·5 per cent. of copper or 0·5 per cent. o copper with 0·5 per cent. of chromium. The tendency to pitting wa very pronounced, being probably aggravated by the large quantitie of dust. The local attack reached 0·5 millimetre in depth after year's exposure whether the steel contained copper or not. Th general attack in the tests was of the order of 0·025 millimetre only so that the problem of pitting was the more important.

(*c*) *Chromium steels.*—Tests given in the Paper showed that th use of steels with a high percentage of chromium (13 per cent., o 18 per cent. with 8 per cent. nickel) was specially to be recommende for protection against corrosion in salt air, but was not at all satis factory for immersion in salt water. The liability to pitting of suc steels in contact with salt water was particularly marked, and wa not reduced by polishing. A higher chromium content (12 t 14 per cent.) reduced the general attack in salt air to 4 to 5 per cen of the amounts for ordinary steel, though, as the Authors pointe

Professor Portevin.

but, the rusty appearance usually gave a wrong impression; pitting was slight, even insignificant. Even with small additions of chromium the attack in salt water was diminished.

For comparison the results of the laboratory tests carried out at Pompey were detailed in Table XLII, the results being expressed as the loss of weight relative to that of the standard steel (of the composition given previously) which was taken as 100.

TABLE XLII.

	Tests in saline mist at $3\frac{1}{2}$ months.	Inter-mittent-immersion tests at $2\frac{1}{2}$ months.	Depth of pitting with intermittent-immersion tests.
Steel with 1 per cent. Cr.	85	85/80	0·05 to 0·1 mm.
" 2 "	74/70	50/55	
" 3 "	60/55	45/50	
" $4\frac{1}{2}$ "	50/45	32/35	0·3 to 0·4 mm.
" 13 "	1·9	2·3	0·2 to 0·3 mm.
" 18 " and 8 per cent. nickel	0·3	0	—
Standard steel	100	100	—

Those tests showed the favourable action of small additions of chromium.

Some previous studies (by Dr. Speller, Messrs. Herzog and Chaudron, and Professor Portevin) had established a reduction in the corrosion of steel in saline solutions by the addition of 2 to 3 per cent. of chromium. The tendency to pitting in intermittent-immersion tests was small with 2 to 3 per cent. but very evident with 4 to 5 per cent. of chromium. In salt mist the tendency to pitting was not more pronounced than with ordinary steels. The difference between laboratory tests and practical tests was especially marked in the case of the steels with 13 per cent. of chromium and with 18 per cent. of chromium and 8 per cent. of nickel.

The general corrosion of chromium steels was insignificant in laboratory tests; in practical conditions it was fairly high when in contact with sea-water. The conditions of use had a large effect upon their resistance to corrosion. The Authors indicated that sheets of 18/8 chromium-nickel steel had a high resistance in clean water in movement. It would seem, therefore, that good aeration was an essential condition for the stability of the protecting film of chromium steels. If parts of that film should disappear through lack of oxygen (due, for example, to organisms) or by any deposit preventing access of the air, the attack would proceed rapidly, and would result in a tendency to pitting at those places.

Professor Portevin.

(*d*) *Ordinary steels and rolled irons.*—According to the results the Authors, the composition of the various materials played on a secondary part in total-immersion tests, the composition an temperature of the sea-water being of greater importance. In sea air the temperature of the testing station was the dominant facto The coldest climate (Halifax) gave the minimum and the hotte (Colombo) the maximum attack.

For specimens immersed under half-tide conditions the influence were more complicated; it was not yet clear what factors came in play. It would appear that the differing periods of immersion an exposure and the height of the tides were concerned. At Halifa the action of molluscs was marked by an inclination to pitting, bu in general, the formation of pitting showed such anomalies that defied interpretation.

For comparison with those conclusions, the following results ha been obtained at the Pompey laboratory. The rhythm of immersio and exposure had an influence on the attack; Table XLIII gav some new figures confirming those published in 1934.[1] The com position of steel would be seen to be of importance.

Table XLIII.—Duration of Immersion equal to that of Exposure— Temperature of 30° C.—Atmosphere Continually Agitated.

	Loss in grams per square metre in 15 days: Rhythm of			Difference per cent.
	10 seconds.	30 minutes.	6 hours.	
Standard Martin steel . . .	278	285	375	33
Steel with— 2·3 per cent. Cr . . . . 1·3 ,, Al	80	130	153	90

The importance should be emphasized of two other importan factors for the aeration and the renewing of oxygen; namely, th stirring-up of the solution and the speed of the wind. The stirring-u of the solution increased the speed of corrosion; for example, a 30° C. the standard steel lost:—

168 grams per square metre after 30 days with 20 stirs per minute.
240 ,, ,, ,, 60 ,,
60–70 ,, ,, ,, without stirring.

The total corrosion diminished by 20 to 25 per cent. with rapi drying of the test-piece after each immersion. On the other hand rapid drying increased the tendency to pitting; a pitting of 0·10 t

[1] See footnote (1), p. 634.

0·15 millimetre in depth was obtained with standard steel after 30 days of intermittent tests, whereas with slow drying it was attacked uniformly. Professor Portevin.

In natural conditions, currents and winds certainly played a considerable part. The resistivity of the products of corrosion varied greatly with the degree of humidity. According to Mr. Herzog, dry rust had a resistivity of 200,000 ohm-centimetres, whilst wet rust had only 1,000 ohm-centimetres. The hygroscopic properties of the products of corrosion had to be considered, as had been shown by Mr. J. C. Hudson.[1]

Laboratory tests in sea-water did not show appreciable differences between Martin and Thomas steels and Armco iron. A high sulphur-content (0·18 per cent.) did not increase the attack; 0·3 per cent. of phosphorus lessened the attack by 20 to 30 per cent. (Table XLIV).

Table XLIV.—Tests carried out at Pompey on the Corrosion of Ordinary Steels: Loss of Weight in Grams per Square Metre.

Steel.	Essential elements: C.	P.	S.	Salt mist, 30 days.	Intermittent immersion, 20 days.
Thomas, before addition of ferro-manganese . . .	0·03	0·08	0·036	197·5	349
Thomas, after addition of ferro-manganese, and poured into ladle . .	0·04	0·06	0·034	194·5	341
Standard Martin . . .	0·08	0·02	0·018	199·0	309
Armco iron	0·02	0·014	0·009	202·0	331
Cutting steel	0·09	0·06	0·176	197·0	328
Steel for screws	0·05	0·35	0·032	181·0	237

In an acid medium sulphur and phosphorus considerably increased the attack, which was increased ten times by a rise from 0·02 to 0·20 per cent. The study of couples showed that iron sulphide and iron phosphide (26·1 per cent. P)[2] gave intense currents of 0·12 to 0·16 ampere per square centimetre in a sulphurous medium, and weak currents of 0·001 to 0·004 ampere per square centimetre with a neutral normal solution of KCl, in accordance with a recent research in the Pompey laboratory.[3] The opinion of Skapski and Chyzewski that manganese sulphide would be more active than iron sulphide

[1] J. C. Hudson, "Atmospheric Corrosion of Metals," Third (Experimental) Report to the Atmospheric Corrosion Research Committee, Trans. Far. Soc., vol. 25 (1929), pp. 177 and 475.

[2] U. R. Evans and T. P. Hoar, "The Velocity of Corrosion from the Electrochemical Standpoint, Part II," Proc. Roy. Soc., vol. 137 (1932), p. 343.

[3] Footnote 1, p. 634.

Professor Portevin.

appeared to be deduced from electro-chemical measurements, whi were liable to error on account of bad contacts at their non-metal electrodes; that appeared to be inevitable with the conditions us when testing. It was for that reason that they found very wea currents (10^{-6} ampere) with iron sulphide, which was a goo conductor. It was rather the beginning of corrosion which wou be increased by the sulphides, as Messrs. Mears and Evans [1] ha indicated.

The Authors' tests at Birmingham showed that sulphides increase atmospheric corrosion. That was in agreement with theoretic investigations, which suggested that iron sulphide ought to increa the attack in an acid atmosphere as found at Birmingham.

(*e*) *Effect of other additional elements.*—In addition to the elemen referred to, there are others which, when incorporated with stee furnish interesting results in connection with salt corrosion, at an rate in laboratory tests. In particular, the addition of aluminiun silicon, and especially the association of chromium with silicon an chromium with aluminium were of interest, and had given th following results :—

	Relative losses of weight.	
	Alternating tests.	Salt mist.
Standard Martin steel	100	100
Steel with 1·2 per cent. Al	60	75
,, 2·6 ,, ,, ...	35	45–50
,, 3·2 ,, ,, ...	25–30	35–40
,, 12·0 ,, ,, ...	5	8
,, 2·2 ,, Si	80	85
,, 3·6 ,, ,, ...	70	75
,, (2+1) per cent. Cr+Si . .	40–45	50–60
,, (2+1) ,, Cr+Al . .	25–30	30–40

Messrs. Sykes and Bampfylde [2] had given the results of tests i salt mist of a series of aluminium steels containing up to 50 pe cent. of aluminium. The improvement was clear, but difficult t compare with Professor Portevin's results, in view of the fact tha they had used various terms of comparison, sometimes steel wit

[1] R. B. Mears and U. R. Evans, "The 'Probability' of Corrosion," Tran Far. Soc., vol. 31 (1935), p. 527.

[2] C. Sykes and J. W. Bampfylde, "The Physical Properties of Iro Aluminium Alloys," Journal Iron and Steel Inst., vol. cxxx (1934, Part I p. 389.

0·8 per cent. of aluminium, at other times with 1·12 per cent. of aluminium, and again with 5 per cent. of nickel. Professor Portevin.

Dr. F. N. Speller, of Pittsburgh, observed that some significant data on the Delhi Iron Pillar were presented on p. 101, which confirmed the conclusion that he and others had previously expressed; namely, that that unique example of ancient craftsmanship in wrought iron possessed no inherent qualities that would make it more durable than modern wrought iron. Dr. Speller had examined and tested a small sample of Egyptian wrought iron, with the same results; it was a clasp taken from an Egyptian monument said to be 2,000 years old, and contained some copper. When exposed, together with modern wrought iron and open-hearth steel, to outdoor atmosphere, humid atmosphere, and aerated hot water, it showed the same behaviour as modern copper-bearing iron. Dr. Speller.

A number of forged iron armour-scales, dating back to A.D. 240, had recently been obtained. They were unearthed at Dura-Europos, on the river Euphrates, by the Yale University Archæological Expedition in 1929, and most of them were only slightly corroded. They measured about 3 inches by $1\frac{3}{4}$ inch by 0·022 inch thick. Test-pieces 1 inch by $1\frac{1}{4}$ inch were cut, surface oxides removed, and the specimens insulated and suspended in Pittsburgh City water heated to 160° F. Eight specimens each of three modern wrought irons and eight each of three basic open-hearth steels, 0·14 per cent. carbon, of the same dimensions, were also included. The specimens tested had the following percentage composition:—

	C.	Mn.	S.	P.	Si.	Cu.
Dura-Europos*	0·09	0·01	0·007	0·114	0·013	Nil
Wrought-Iron .	0·03	0·06–0·07	0·019–0·021	0·153–0·160	0·20–0·25	0·020–0·024
Steel . .	0·13–0·14	0·54–0·55	0·027–0·028	0·006–0·008	0·20–0·23	0·012–0·016

* The Dura-Europos sample was free from Ni and Cr.

After 3 months in the water some of the ancient iron samples were perforated, so the test was discontinued and the specimens weighed. Stated as average penetration in inches per year, the loss in the three types of metal tested was:—

Steel	0·0140
Modern wrought iron	0·0143
Dura-Europos iron	0·0137

The ancient specimens of iron were much more heterogeneous in structure than modern wrought iron, but showed no essential

Dr. Speller.

difference in durability. When cleaned and exposed to t atmosphere, they rusted rapidly and lost far more in 3 months water of average domestic quality, as indicated above, than th had suffered in 1,700 years of exposure to the dry conditions whe they were found. Further details in regard to the metal had bee given elsewhere.[1] The quantitative tests described substantiat the conclusions expressed by Sir Robert Hadfield and other inves gators that the remarkable durability of certain specimens of ancie iron was due to their favourable environment, and not to an superior inherent durability.

More recent tests, conducted by the American Society for Testi Materials in various waters, and by the National Bureau of Standar in forty-seven different types of American soils, indicated no materi difference in corrosion-resistance between wrought iron and t various grades of commercial pure irons or steels under those co ditions. The differences in life of those metals in service under su conditions had evidently been controlled mainly by factors extern to the metal. However, it was found recently that certain low alloy-steels designed to form more resistant films in certain kinds water or air showed superior durability.

Colonel Trinham.

Colonel J. S. Trinham observed that the Paper was of speci interest to manufacturers and users of genuine wrought iron made t the puddling process. There were still nineteen or twenty firms i Britain making good wrought iron in considerable quantities, an that iron would give results in service as satisfactory as thos mentioned in the Paper. The 36-per-cent. nickel steel and th high-chromium stainless steels, though of high corrosion-resistanc were naturally beyond practical comparison with wrought iron, they were commercially expensive, but what might be called th "general commercial mild steel" was in a comparable category.

The data given by the Authors relative to the distinctly differen types of corrosive attack found to have occurred during the lon duration tests of iron and steel samples in conditions of aerial, hal tide and total-immersion attack at the four posts geographicall wide apart were of particular interest. They had made it possib to compare the behaviour of wrought iron, ingot iron, and mild stee in their relative resistance to corrosion, general and local, mo closely and more satisfactorily than had hitherto been practicabl From those data he had tabulated the estimated mean depths general corrosion, waste, and pitting produced so as to indicate th resistance to corrosion of the ingot iron, mild steels, and copp steels relatively to that of wrought iron. In Table XLV the relati

[1] *Metals and Alloys*, August, 1936. [New York.]

resistances, taking that of the wrought iron as 100, were estimated on the basis that the depths of corrosion attack were inversely proportional to the resistance to corrosion possessed by the corroded metal. Colonel Trinham.

TABLE XLV.—MEAN DEPTHS OF GENERAL WASTAGE AND OF DEEPEST PITS: AVERAGES FOR ALL FOUR PORTS.

Material.	General wastage: millimetres.	Relative resistance.	Maximum depth of pitting: millimetres.	Relative resistance.
Wrought iron N	0·499	100·0	1·16	100·0
Swedish charcoal iron P	0·573	87·1	1·35	86·2
Ingot iron M	0·664	75·2	1·62	71·5
Mild steel B, 0·21 per cent. carbon	0·617	80·8	1·91	61·0
Mild steel F, 0·24 ,, ,,	0·607	83·0	2·19	53·0
Mild steel E, 0·34 ,, ,,	0·630	79·4	2·10	55·2
Steel D, 0·40 ,, ,,	0·599	83·3	2·16	53·8
Copper steel G, 0·21 per cent. carbon with 0·63 per cent. copper	0·499	100·0	2·18	53·2
Copper steel H, 0·22 per cent. carbon with 2·19 per cent. copper	0·472	105·3	1·96	59·2

The authoritative results given in the Paper, and arranged in comparative form in Table XLV, corroborated previous findings to the effect that genuine puddled wrought iron resisted ordinary corrosive attack more satisfactorily than mild steel, ingot iron, or Swedish iron, and was much less prone to the far more serious "pitting" action experienced in both underground pipe service and marine practice. The Authors had duly recognized the seriousness of the pitting type of corrosion, and had shown in Table XXIV (p. 88) that even in the stainless type of chromium steel the deepest pits had a mean depth of 1·74 millimetre in the averaged tests (aerial, half-tide, and total immersion) whilst in the wrought iron (Table XIV, p. 78) the mean depth was only 1·16 millimetre. In the case of the copper steels it would be noted that, whilst the depth of the general corrosion wastage approximated to that of wrought iron, the depth of the average maximum pitting was from 69 to 88 per cent. more than that of wrought iron. It was interesting to read that "the Sea-Action Committee's research had clearly shown that, for conditions such as those of total immersion in the sea, alloy steels, while in general showing a superiority over plain carbon steels, are not so effective in reducing wastage as they are under half-tide or aerial conditions."

Table XXXVII (p. 106), given by Mr. Wilson, did not bring out the superior corrosion-resisting properties of wrought iron, as ingot iron, Swedish iron, and wrought iron were grouped together, giving the relative corrosion figure of 112, whilst the adjacent figure for steel

Colonel Trinham.

was also 112. Separately considered, the wrought iron had show less general wastage and less pitting than the other irons with whi it was grouped.

If, as shown in the foregoing Table, the depth of the local pittii and of the general corrosion wastage was less in wrought iron tha in ingot iron or mild steels of the plain carbon type, there would ha to be, as a result of the authoritative findings of the Institution's Se Action Committee's research, a recognition of the fact that genui wrought iron retained its character of resisting sea-water corrosi in the widely different conditions operating in the four ports and f long periods. Moreover, the tests proved that wrought iron ha withstood such corrosive attack more satisfactorily than ingot irc or mild steels. As the wrought-iron samples were descaled befo the tests, further tests might show wrought iron in a still better ligh

Dr. Tutin.

Dr. JOHN TUTIN observed that such a comprehensive Paper wou be of great interest and value to the shipbuilding and shipping in dustries, particularly as ships represented a very large proportion the world tonnage of steel exposed to sea-action. Even an ordinar tramp ship of 9,000 tons deadweight might carry as much as 200 ton of steel in excess of that required for normal structural purposes, solel to provide a margin of material available for destruction by th various processes which were detailed in the Paper. For that burde the shipowner had to pay not only the extra first cost of material an labour (roughly £3,000 for the small vessel in question), but also th annual cost of propelling 200 tons of otherwise useless materia which also robbed him of the freight earnings on 200 tons of carg for the entire life of the ship. The corrosion problem, therefor justified constant attention by naval architects if the shipowner expenses in that direction were to be mitigated, and the presen research embodied the results of observations on such a wide rang of specimens, under such a wide range of conditions, that it ranked a one of the most important single contributions ever made to th subject. Attention had been drawn to the progress of the researc in a Paper entitled "The Corrosion Problems of the Nav Architect,"[1] but, unfortunately, in that Paper the Institution c Civil Engineers' programme of research had been referred to as "fallin short of the ideal" in several respects, so that it might carry les conviction in shipbuilding circles than it was entitled to. Fc example, it was stated that "duplicate specimens were not exposed. From the Paper now under discussion it was seen that not onl duplicate but triplicate specimens were provided for the whole c

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[1] W. H. Hatfield, "The Corrosion Problems of the Naval Architect," Tran Inst. N.A., vol. lxxvii (1935), p. 158.

the main series of tests. Furthermore, it had been stated that the effect of scale on the specimens had not been properly taken into account, particularly for the chromium steels. It was, however, evident from the present Paper that throughout the tests adequate provision had in fact been made in that respect. Dr. Tutin.

Mr. H. E. YERBURY was gratified to observe that the line of demarcation originally defined between chemical and electrical processes was nearly broken down. In his opinion the subject of corrosion in general could better be understood when electrical processes entirely supplanted what was called "chemical action." Nothing in the Paper falsified the views which he had previously expressed.[1] From a practical view-point, there was no doubt that corrosion of ferrous metals had increased greatly since steel—as used in commercial work—had replaced iron. All commercial steels, whether Siemens or Bessemer, had manganese added for the sake of good working under the press, and that greatly increased their tendency to corrode. Unfortunately alloy steels were too expensive for heavy engineering work, with the result that more attention was being paid to the protection of structures from corrosive influences. The simple fact remained that, whatever medium was employed as a coating, if it remained impervious to moisture (an electrolyte) no corrosion could take place. Mr. Yerbury.

The AUTHORS, in reply to the Correspondence, observed that both it and the Discussion had shown the high value which was placed by other workers in the field of corrosion on the data so far provided by the extensive researches of the Sea-Action Committee regarding the corrosion of iron and steel. That, they felt sure, would be taken by the members of the Committee as some reward for their labours of the past 20 years. The Authors.

They were pleased to find that their Paper had evoked such a wide discussion, in which many of the most prominent authorities on corrosion had taken part. The contributions from abroad were especially welcome; giving as they did authoritative and up-to-date accounts of the progress in the subject being made in their respective countries, they would be found particularly helpful.

Several contributors had taken the opportunity afforded of comparing the results of their own laboratory tests with those of practical exposure-tests, and it might be said that on the whole the degree of agreement was most encouraging. Not many years ago such agree-

[1] "The Effect of Air and Water on Materials used in Engineering Work," Supplement to vol. 57, Journ. I.E.E. (1919), p. 118. "The Electrolytic Action of Return Currents in Electric Tramways on Gas- and Water-Mains; and the Best Means of Providing against Electrical Disturbances," Min. Proc. Inst. C.E., vol. cc (1914–15, Part II), p. 62.

The Authors. ment was uncommon, for two reasons: firstly, that authentic da on corrosion under practical conditions for an extended period, su as those now provided by the Committee, were scarce; secondl that the same effective control was not then exercised in laborato tests as was now the case. It would appear from the degree agreement now obtained that the effects of at any rate the princip factors operating, including temperature and oxygen-supply, we becoming better understood, and that the field for further invest gation was being narrowed to those cases where important differenc occurred between the results of laboratory and practical exposur tests. Although therefore, as Dr. Desch pointed out, the complexit of the process of corrosion was becoming more apparent, the Autho did not take the rather pessimistic view of Mr. Bennett that th causes of corrosion appeared to be more elusive than ever. To suc a degree, in fact, had knowledge advanced that in the plannin of subsequent researches of the kind undertaken by the Sea-Actio Committee it had been found necessary to take account of many mor factors, both in the manufacture of the test material and in th environment during exposure, as explained by Dr. Hatfield wit regard to the researches being made by the Joint Committee of th Iron and Steel Institute and the British Iron and Steel Federatio

The special attention paid by the Paper to the particularly viciou form of pitting corrosion suffered by the high-chromium steels i sea-water had brought useful comment and suggestions from number of correspondents. The Authors agreed with Dr. Hatfiel that the 18—8 chromium-nickel type of steel might behave excel lently in sea-water where oxygen shielding was not a serious facto but unfortunately such conditions could not be relied upon excep in rare cases, especially with stationary structures. Local restrictio of oxygen-supply might be caused in many ways by natural agencie such as molluscs and other marine growths, or by the settling of an particles in suspension on to the surface of the steel. Also, thos immersed steel members which were necessarily in an upright positio were not so accessible to the aerial oxygen at their lower as at thei upper portions. Even in the most favourable cases, to ensur uniform supply of oxygen the avoidance of corners and crevice was involved, thus putting serious restrictions on design.

There seemed to be much evidence in favour of the explanatio of the serious form of pitting put forward by Dr. Evans and base on actual measurements of anodic voltage and current. Vigorou and localized electrical action did seem to be at work. The pro tective film which was characteristic of the modern "stainles steels," and which gave them such excellent and useful propertie for many purposes, was an armour which was vulnerable unde

ertain forms of attack, namely, where anodic conditions were set up in the presence of a suitable electrolyte, such as sea-water. All explanations pointed to chromium as being the basis of that protective film, and the Authors' suggestion that the comparatively low chromium-content of specimen No. 3765 (chromium 10 per cent., nickel 20 per cent., tungsten 2 per cent., copper 2 per cent.) helped to account for its superior behaviour as regards pitting in the tests conducted in the Gulf of Paris was well borne out by the results of other exposure-tests. Passivity of the surface caused by pickling, as suggested by Dr. Hatfield, could not be expected to operate for any length of time under such conditions; the chief effect of pickling was to be attributed to the uniformity of surface which it effected by the removal of all scale. The Authors.

The appearance of the chromium and nickel-chromium "stainless" steels at the onset of pitting had often given a strong impression that the pits were formed at centres provided by non-metallic inclusions, but that impression might now be finally dismissed. Apart from the fact that such steels were among the cleanest made, close investigation of cases of pitting in their initial stages had been made in the Authors' laboratories, and in no case had any connection with inclusions been established.

With the suggestion of Dr. Desch that the mechanism of pitting in the highly-alloyed steels should be further studied, the Authors, who had experimental work in hand on that question, heartily agreed. Regarding the adhesion of barnacles, experience in the Committee's researches seemed to show that it was not materially influenced by the composition of the steel.

The possibility of biological action by molluscs promoting corrosion had been mentioned both in the Committee's reports and by the Authors; present evidence seemed to show that it was not an important factor in most cases. Dr. von Wolzogen Kühr called attention, however, to the importance of bacterial action in promoting corrosion in certain cases where attack would not ordinarily be expected, owing to the exclusion of atmospheric oxygen. The Authors understood that such "anaerobic" corrosion, although occurring more particularly on iron and steel buried in soil, had been associated also with the attack on piles of piers where embedded in the mud of river-beds. Sulphide of iron was one of the products of the bacterial action, and the interesting information provided by Drs. R. B. Mears and U. R. Evans and referred to by Professor Portevin as to the part played by that compound in corrosion-processes might therefore be usefully studied in that connection.

With regard to the rather thorny subject of the relative merits of wrought iron and ordinary steel, the Committee's programme

The Authors. intentionally included specimens for the purpose of providing reliable comparative evidence on that question. While wrought iron of good quality was shown to behave rather better than ordinary steels, the superiority was not very pronounced, and further, it could not be hoped by such means to set up a court of appeal, to which protagonists of steel or iron could refer in settlement of their friendly differences! It was shown that the relative performance depended on environment, and the variety of conditions under which those materials were actually employed was too wide for any one research to cover. Nevertheless, the practical evidence provided on the question by Mr. M. F-G. Wilson,[1] Dr. Speller, Colonel Trinham, and Mr. Yerbury would be read with interest.

It was probably true that, as mentioned by Dr. Desch, wrought iron owed something to its texture or fibrous character resulting from slag. The characteristic appearance of corroded wrought iron certainly seemed to indicate a directive influence of the slag on the corrosion. Texture as well as chemical composition was in many cases an important factor in the behaviour of metals, and the Authors were reminded of Dr. Desch's instructive lecture on that subject,[2] which was well worth studying.

With reference to Mr. Yerbury's remarks in regard to the possible influence of manganese in comparatively small percentages (up to about 1 per cent.), researches by many authorities did not seem to show it to have any material influence on the corrosion of ordinary steel, or, at any rate, to be of such prominence as to prove the determining factor in its characteristics as compared with those of wrought iron.

In reply to Professor Honda and Dr. Endo, the Authors were unable to make any suggestions in explanation of the contrasting behaviour of iron and steel with regard to pitting under half-tide conditions at Colombo, other than those given in the Paper.

It was pointed out by Professor Honda and Dr. Endo that 0·35 per cent. was the limit of the solubility of copper in iron at ordinary temperature, and the inference might possibly be drawn that no more than that amount could usefully be added. It should, however, be noted that that percentage applied to conditions of complete equilibrium in the steel, and that ordinarily more than that amount would be found in solution. Even, therefore, if solution of the copper were a necessity in order that it should improve resistance to corrosion (which was by no means established), there was no anomaly in the fact that larger percentages than 0·35 had been found to be

[1] Discussion, p. 104 (June).

[2] "Texture and Chemical Resistance," Trans. Inst. Chemical Engineers, vol. 12 (1934), p. 198.

advantageous in the Committee's researches. It had recently been stated that a proportion of the wagons being ordered by the London, Midland, and Scottish Railway and the London and North-Eastern Railway would be built from steel containing 0·25 to 0·45 per cent. of copper. It would be particularly interesting to learn in due course how they behaved in service. The Authors.

Special attention should be called to the useful practical information placed at the disposal of engineers by Dr. Footner regarding experience with pitting, and methods of pickling to remove scale.

The Authors were indebted to Dr. Tutin for his remarks on the thoroughness with which the Committee's researches had been planned, and particularly on the fact which those studying the subject should bear specially in mind, namely, that adequate provision had been made in the way of duplication of specimens and in the inclusion of specimens for the investigation of the effect of scale.

The interesting information which he gave regarding corrosion and its economic bearing on ship-construction was an indication of the importance of the subject.

One of the Authors showed [1] in 1922 that the annual losses from corrosion, at a conservative estimate, amounted to the large sum of £700,000,000. Whilst the researches by the Sea-Action Committee and by the Joint Corrosion Committee of the Iron and Steel Institute and the British Iron and Steel Federation—in which two Committees, with 43 individual members, were at work—and also other organizations throughout the world, had not as yet solved the problem completely, yet the increased knowledge brought to bear was certain to prove of the greatest value in its economic results. Whilst the labours of the Sea-Action Committee were largely drawing to a close, those of the Iron and Steel Institute continued, and all would wish every success to their efforts. If proof were needed of the widespread importance of the subject, the many valuable contributions to the Discussion and Correspondence from Great Britain, the United States, France, Japan and Holland, would provide it in full.

In conclusion, the Authors would like to express their thanks to the contributors for their appreciative remarks in the Paper, and especially to Dr. Newton Friend, whose kind remarks regarding their very friendly relations in the work allotted to them in the Committee's Research programme were most welcome and were heartily reciprocated.

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[1] Sir Robert Hadfield, "Corrosion of Ferrous Metals," Minutes of Proceedings Inst. C.E., vol. ccxiv (1921–22), p. 83.

Paper No. 5058[1]

"Some Developments in Railway-Carriage and Wagon Construction."

By Paul Lewis Henderson, Ph.D., B.E., Assoc. M. Inst. C.E., A.M.I.Mech.E.

Correspondence.

Mr. Hendriksen.

Mr. E. E. Hendriksen observed that it was satisfactory to se welding so well used in railway rolling stock. The constantly varyin stresses and strains set up by loadings and vibrations would give th work a thorough test. Many designers were concerned with th effect of time on the strength of a weld and of the neighbourin parent metal, and it was advantageous that in the work describe the joints were open to continual inspection; the reports on the behaviour would be of value.

He wished to refer particularly to the welded connection of a stee section framing on to the plane face of a cross member. The calcula tions for the design were given on pp. 252 and 281. The bendin moment at the end of the steel section was resisted by each part of th weld, and in proportion to its distance from the neutral axis. Th use of the full section of the weld in finding the moment of inertia and hence the stress f_2 due to bending, was theoretically correct it was to be noted that in the examples given the stress was increase by $17\frac{1}{2}$ per cent. if the weld-metal adjacent to the flanges only wer used in obtaining the stress from bending. That was larger than i most girder-designs, on account of the large effective thickness of th web of the weld-metal being equal to the flange thickness.

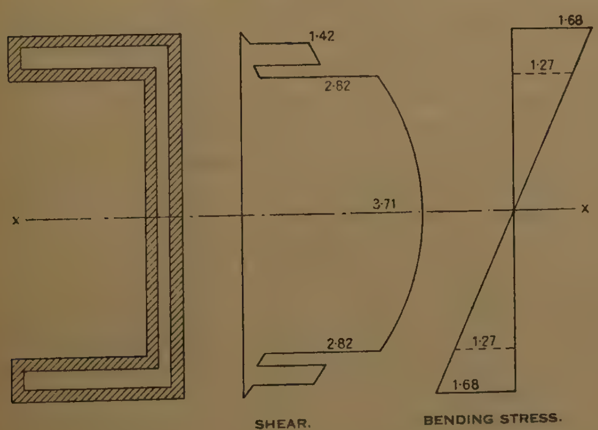
The question of shear stress in the weld-metal was of a more com plex nature. In the first example, the Author had taken the shea over a depth of web equal to the clear distance between the flang welds, whilst in the example for test purposes (p. 281) the clea distance between the flanges of the steel section had been used. Th shear stress f_1 was given as 3·58 tons per square inch for the secon method, whereas if the former method were applied it would b 3·92 tons per square inch, an increase of $9\frac{1}{2}$ per cent.

[1] p. 231 (June).

It was well known that the actual shear stress in a member under flexure varied from zero at the extreme fibres to a maximum at the neutral axis; in a homogeneous rectangular member the distribution was parabolic, with a maximum of one-and-a-half times the average over the full area. In a large steel joist (with substantial flange areas), the shear distribution was fairly uniform over the clear depth of the web, and a very small portion was taken by the flange. In the examples under consideration, there were virtually two flange and web pieces placed adjacent, but with different clear distances. The shear stresses were somewhat complicated, and for clearness the test-example was shown in *Figs. 37* with the shear and bending

Mr. Hendriksen.

Figs. 37.



stresses. It would be seen that over the clear web depth, the shear varied from 2·82 to 3·71 tons per square inch. The Author's value of 3·58 was therefore only a little below the maximum. The weld-metal at any point was thus subject to equal horizontal and vertical shear stresses, and to a horizontal direct stress due to bending. The resulting condition was one of principal stresses of pure tension and compression, and a maximum shear stress. Those were obtained by the method indicated on p. 253, namely:

$$\text{Maximum principal stress} = \frac{f_2}{2} + \sqrt{\frac{f_2^2}{4} + f_1^2}$$

$$\text{Maximum shear stress} = \sqrt{\frac{f_2^2}{4} + f_1^2},$$

where f_1 and f_2 denoted the shear and bending stresses at the point under consideration.

Mr. Hendriksen.

If that were applied to the figures given in *Figs. 37*, it would b found that the stress-conditions were :—

Extreme fibres :	principal stress of	1·68	tons per square inch,	shear stress	nil
Edge of web :	„ „	3·53	„ „	„ „	2·
Neutral axis :	„ „	3·71	„ „	„ „	3·

The stresses at the edge of the web were due to a combination of shear stress of 2·82 tons per square inch with a bending stress 1·27 ton per square inch.

It was evident that the governing factor in the strength of th connection was the shear stress of 3·71 tons per square inch at th neutral axis. Taking the ultimate shear strength of the weld meta at 18 tons per square inch, it was clear that the probable ultimat load would be at least 11 tons $\times \frac{18}{3 \cdot 71} = 53 \cdot 4$ tons. As there woul no doubt be a certain redistribution of stress in the metal just befor failure, a higher load could be expected, and that was indicated b the test. Further, since the shear stress alone was the governin factor, the two examples ought to be expected to give similar result That was verified by the closeness of the test loads, 58·0 and 58·4 tons That also indicated the uniformity in fabrication of the two welde connections and spoke highly for the operators. It would b instructive if observations at the time of testing could throw an light on the manner of failure. The considerations given abov would be too laborious for ordinary design purposes, but the indicated the probable condition, and might lead to simple rules o criteria for regular use.

In the examples of the welding tests, it was interesting to notic that the average factors of safety with the adopted design stresse were

End fillet,	stressed at	5	tons per square inch :	factor of 5.3
Side	„ „	4	„ „ „ „	4·5
Butt	„ „	5	„ „ „ „	5·7

It was obvious that a more uniform factor of safety was desirabl and hence the design stresses could well be readjusted.

In the test on the joint subject to bending and shear, it was o interest to note that if the shears and bending stresses given wer combined in the manner previously indicated, the result would b principal stresses of 4·8 and 4·5 tons per square inch in the tw cases, and a shear stress of approximately 3·7 tons per square inch in each case. An increase of the load to 12 tons would bring those values to approximately the design stresses, and using an averag factor of safety of 4·9 (that was, between 4·5 and 5·3), the test loa would be 12 × 4·9 = 59 tons. That agreed with the actual test

although it was based on the incorrect procedure of combining an average shear stress with the extreme fibre tensile stress. Mr. Hendriksen.

The Author, in reply, observed that he was very interested in Mr. Hendriksen's comments although he could not agree with all of them. It was certainly of great value to designers to know the effect of time on the strength of welds; reports to date on the welded work described in the Paper had not so far disclosed any weakness or failures of welds, although the vehicles concerned had been in constant service. The Author.

With reference to the remarks regarding the method of designing a welded joint subject to bending moment and shear, given on pp. 252 and 253, it had to be remembered that the method was stated to be only "approximate, owing to the several assumptions made" (p. 254). The method had the advantage of being quickly applied, and was intended for ordinary design-purposes, as it gave a safe upper limit. Admittedly, it was theoretically correct in the case of steel sections to take the full section into account in finding the moment of inertia and hence the stress due to bending, but it did not necessarily follow that the same procedure was to be adopted in an approximate method for determining the principal stresses for designing a weld similar to *Fig. 17* (p. 252). In that connection the following statement by Professor Morley in his book on the "Theory of Structures," p. 150, would be of interest:—

"*Principal stress in* I *sections.*—In I sections, whether rolled in one piece or built up of plates and angles, it has been shown . . . that the web area is of little importance in resisting the longitudinal direct stresses due to bending, or, in other words, it contributes little to the modulus of section, and . . . the flanges carry little of the shear stress."

In most cases it might be sufficiently accurate, when determining weld dimensions, to assume that the flange-welds took the bending stresses and the web-welds the shear stresses, even although the stress might be, as in the present case, increased by 17 per cent. on the safe side. It was a different matter when calculations underestimated stresses; Professor B. P. Haigh, however, had shown[1] that the yield loads for straight beams of channel sections, in certain cases ". . . were found to be only 43 to 80 per cent. of the calculated F value, according to the definition of the yield-point adopted," the usually accepted methods of calculation being used. It would be noticed that the maximum shear in *Fig. 37* was 3·71 tons per square inch, whereas if the shear were considered to be evenly

[1] "Constructional Tests on Mild-Steel Rolled Sections with Electrically Welded Joints." Trans. Inst. N.A., vol. lxxv (1933), p. 59.

The Author. distributed over the clear distance between the inside flange-welds the value obtained would be 3·89 tons per square inch (not 3·9 as stated by Mr. Hendriksen), which had the merit of being much more quickly determined and was also on the safe side. The distance between the welds of 6·16 inches used in the example on p. 281 was an oversight, and should have been 5·66 inches.

He agreed that *Figs. 37* was correct from theoretical considerations, as it was arrived at by the usual methods ; the shear diagram had been obtained by calculating the horizontal shear values as done on p. 249 of the Paper, and the bending-stress diagram was easily drawn, with the value of 1·68 ton per square inch for the extreme fibre stress, as given on p. 281, example (ii). The method shown in *Figs. 37* should be used for any special cases, but it was, perhaps, rather laborious for ordinary drawing-office use. He rather questioned the statement that " . . . the shear stress alone was the governing factor . . . " in the failure of the welds tested, as he had tested many welds similar to that calculated on p. 281, but he had not seen one that showed first signs of failure on the neutral axis where the highest shear stress occurred. In all cases the members had commenced to fail at the top-flange welds, the failure gradually or suddenly proceeding downwards.

With regard to the welding tests shown on p. 280, Mr. Hendriksen had suggested that the factors of safety could well be re-adjusted, but the Author wished to point out that the tests shown on p. 280 were typical tests only, and had not been used for determining the factors of safety. Factors of safety for welds were not usually determined by the results of two isolated tests, but rather on the results of many tests, whilst other factors, such as fatigue and impact values, the position in which the electrode was to be used, and the properties of the parent metal, were also taken into account.

EXTRA MEETING.

26 May, 1936.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S.,
Vice-President, in the Chair.

SPECIAL LECTURE ON

"Recent Developments in Metallurgy and Their Influence on Engineering."

By CHARLES-EUGÈNE SCHNEIDER, D.Sc., Member of "Institut de France."

WHEN my distinguished friend Sir Robert Hadfield invited me to take part in your Meeting, I greatly appreciated the honour he was doing me in asking me to place before the premier Institution of engineers in Great Britain my views upon certain questions which are attracting general attention. I greatly appreciated also the kindly thought which led him to select the present year, 1936, for my communication, marking as it does the hundredth anniversary of the taking over of the Creusot works by my grandfather, who greatly and rapidly developed them. During the growth of the Creusot works, we have never ceased to entertain friendly intercourse with the British iron and steel industry—an industry which in 1836 was predominant the world over—and to exchange ideas, results of experiments and discoveries with British metallurgists; we have constantly collaborated with them in evolving new methods of working and improvements in plant. I may, therefore, add that Sir Robert has thus afforded me the opportunity of celebrating to-day a century of friendship, and, in the first place, I desire to tell him, as one who has so greatly contributed to knitting together these bonds of friendship, the gratification I feel for his kind invitation, and to place on record my admiration for his own engineering and scientific work.

The present year also marks an important centenary for your Institution. Over 118 years ago there met at Kendal Coffee House, Fleet street, a small group of men who were fully alive to the future in store for engineering, to the ever-increasing complexity of the problems awaiting solution, and to the necessity for close co-operation between engineers, as many of the intricate problems constantly

arising could not be satisfactorily dealt with by individual effort alone. The year 1836 marked the publication of the first volume of those " Transactions " which were destined to widen in such a large measure the influence of your Institution by recording and disseminating both in Great Britain and abroad the communications made at your meetings. You thus celebrate during this present year the hundredth anniversary of your Proceedings, which form now a whole reference library available to all metallurgists and engineers.

This year is worthy of note from another standpoint, since in 1836, thanks to your celebrated Stephenson, our Séguin, and their followers, the railways commenced their marvellous development which was destined completely to transform our entire civilization. At that early date engineers got themselves ready to cover the whole world with an immense network of railway tracks, roads, canals and oversea shipping lines, and to build bridges, drill tunnels and equip harbours. Industrial, agricultural and commercial activity was distributed in new directions, and greatly influenced politics. Regions and even whole countries specialized in certain classes of work. This led to a rapid growth of population which tended to concentrate in those parts of the world best situated from an economic point of view.

The reorganization of the world I here allude to, in which scientific methods took the place of those of " rule of thumb," required the creation of a new profession, which may be said to have been non-existent previous to the nineteenth century ; the profession to which you, gentlemen, belong, the profession of the civil engineer. The huge future programme here briefly outlined was a vast one for your predecessors of 1836 ; you know in what manner they, their successors, and you who followed them, have met it. A most significant proof of the success achieved in this connection is afforded by the fame of your Institution, by the authority that attaches to it throughout the world, and by the number of its members, which now exceeds eleven thousand.

The views in regard to the economic future of the world which led your founders to form this Institution were also those which influenced my grandfather when he decided to set up his Creusot works. He did not allow himself to be deterred in any way by the lack of success of the personalities or concerns who had preceded him in this endeavour ; he foresaw the extent of future needs in all classes of machinery, industrial products and civil engineering, resulting from new methods of transport. He decided to carry out their manufacture at Creusot and Chalon-sur-Saône, and to combine with it a correspondingly complete metallurgical installation. The intimate connection between metallurgy, engine construction, shipbuilding and civil engineering

has been the constant ruling factor in the activity at Creusot during the last century and down to the present time, as it has continued to be the guiding principle of your Institution. It may therefore be asserted that a state of affairs which was full of possibilities inspired in that same year, 1836, both here and in France, the same ideas in the discerning and practical men who were the founders of your Institution and the founder of the Creusot works. It appeared to me that I could honour the memory of our predecessors in no better way than by bringing out the successful results of the principles that all adopted and by emphasizing also the close connection that has prevailed during the century ending in 1936 between the various branches of engineering, and notably between metallurgy and mechanics, a connection so close that all progress in one branch is immediately reflected by progress in the other. I do not claim for one moment that I am about to state anything new, nor is this lecture going to be highly technical or scientific, but it will simply be a review, limited to historical facts, and calling up useful thoughts when they are examined in the light of present knowledge.

In the first place I consider it a pious duty on my part to pay a tribute of admiration and gratitude to the men who have brought honour to your Institution, notably those who were its Presidents during the last century. The list opens with a remarkable man, Thomas Telford, born of peasant parents, who worked as a mason's labourer in his early youth and by his unaided efforts rose from this lowly start in life to one of the highest ranks in English engineering, gaining world-wide repute by his work in road, canal and bridge construction. By burying him in Westminster Abbey, your country paid a solemn homage to his work as an engineer and to his sterling worth.

Then followed the Stephenson family, Robert Stephenson, the son and collaborator of the celebrated locomotive builder, who transformed the railway from an object of mere curiosity into the most powerful means of transport of modern times; and George Robert Stephenson, a cousin of Robert and associated with him in developing railway systems.

Another most brilliant name on the list is that of Lord Armstrong, a man of many parts, who combined in himself the lawyer, mathematician, electrician, economist, administrator, inventor of the hydraulic crane, the hydraulic press, and a new breech-loading gun, and founder of extensive works.

During the remarkable Victorian era, when vast schemes brought great men into life and action, I find at the head of your Institution a whole series of eminent public-works engineers: Sir John Coode, the builder of harbours, among which are Portland, Colombo,

Cape Town, Durban and many others; Sir John Fowler and Sir Benjamin Baker, with whom in the early years of my work I had the great honour of collaborating, and whose names remain specially connected with the London Underground Railway and the gigantic Forth Bridge.

What a number of other illustrious names I might add to the few mentioned above, were I not limited as to time, and had I not decided to confine my review to the nineteenth century, so as not to upset the modesty of some of my hearers! In point of fact, on looking through the annals of your Institution, I came across a long series of names associated with everything great in the way of civil engineering accomplished throughout the world, and I then realized the difficulty of the task which the kind insistence of Sir Robert Hadfield had induced me to accept, of placing before you a communication worthy of engaging the attention of a select body of engineers such as are present here this evening, and who are accustomed to carry out great works and to solve the hardest problems arising therefrom.

A century ago the metallurgy of iron was in a process of evolution; many works were still following methods which differed from those of preceding centuries in matters of detail only. A certain number of forges, however, were commencing to use new apparatus and processes, and these led to improvements which have resulted in the methods now employed in the equipment of our modern steelworks.

Let us consider, for instance, the position of France in 1836; she was then the second metallurgical country in Europe, the first place being occupied by England, whose output of pig-iron and wrought iron was equal to that of the rest of the world. Of the five hundred and forty-three furnaces then in blast in France, five hundred and two were still fired with charcoal. Their average daily output amounted to 3,750 lbs. (1,700 kilos.). Surface ore-pockets containing a few thousand tons of iron ore were sufficient to supply those blast-furnaces over a number of years; most of those deposits have been given up long ago and many of them have been lost sight of.

The geographical distribution of the works was quite different from that prevailing at the present time. The old forges, located near the forests which supplied the fuel, were on the banks of rapid-flowing streams suitable for working by water-wheels the blast-furnace and hearth bellows, and the small mechanical hammers. The manufacture of iron with charcoal was found over the whole country, the principal centres being then in the Departments of Haute-Marne, Haute-Saône and Nièvre, where there are still to-day plants of some importance, although they no longer make pig-iron, their speciality being the working up of metal charges.

The output of the charcoal blast-furnaces was restricted by the

output of the forests, which were drawn upon to their full capacity and were strictly controlled. In such conditions no progress was possible. About 50 years previously—following the English example, but on a reduced scale—attempts had been made to use coal in the manufacture of pig-iron and especially in its refining. The first French coke blast-furnace was that of Creusot built in 1783. This meant turning to account the reserves of fuel accumulated in prehistoric times instead of the wood fuel of 30 to 40 years' growth. This evolution, which introduced the era of "heavy industry," was delayed for a time by a lack of suitable means of transport: the railways were destined to fill this need.

In 1836 the Creusot works owned four coke blast-furnaces, served by a 100-HP. blowing engine; they were charged with poor local ores giving a yield of only 20 per cent., which my firm replaced later by Berry ore transported on the canal leading from the Loire. The coke consumption was never below 1·9 tons (1,900 kilos.) per ton of pig-iron. The annual output of pig-iron was 8,000 tons, equal to 5·5 tons per day for each blast-furnace. It was almost entirely transformed into wrought iron in coal-fired hearths introduced some years previously by the English metallurgists Manby and Wilson. In 1836, my firm produced 7,000 tons of iron in bars and plates, with a staff numbering 2,000.

The works at Alais and Decazeville, which were practically the same in size, had been started a short time before with a view to meeting the requirements of the newly-established railway industry, and, like those of Creusot, were in close proximity to deposits of coal. This meant a new distribution of ironworks, inasmuch as they were becoming independent of forests and rivers.

The manufacture of iron castings also developed rapidly. Coke-fired cupolas gradually replaced reverberatory furnaces for melting, and sand was taking the place of loam for the making of moulds. At that early date foundries commenced the manufacture of heavy castings, such as those for the spire 500 feet (150 metres) high for Rouen Cathedral, the roof supports of Chartres Cathedral and the voussoirs for the arches of the Paris Carrousel bridge, a bridge now in course of reconstruction.

The puddling furnace invented by Cort in 1784 for making iron was replacing the refining hearth, and by 1836 the output of each of these two kinds of furnaces was approximately the same in France. A short time later wet puddling, in which the pig-iron is treated in contact with iron oxide, was introduced. This process was the forerunner of the "ore process" which was employed in so many open-hearth steelworks, as soon as it was known how to construct furnaces capable of melting mild steel.

Several works were equipped for the manufacture of butt-welded iron tubes, required in ever-increasing quantities by gasworks and locomotive boilermakers. A number of improvements, which were to allow of a rapid increase in the output of existing forges, were already in the stage of being worked out at this time. Neilson had been heating the blast supplied to the Clyde blast-furnaces since 1829; whilst, in 1805, Aubertot had started utilizing blast-furnace gas at Vierzon for calcining lime. A blast-furnace in the vicinity of Grenoble, combining both these principles, was equipped in 1833 with an apparatus for heating the blast by means of blast-furnace gas.

The rolling-mill had already been introduced into everyday practice by Cort at the same time as puddling. The steam hammer, invented in 1841 by Bourdon, an engineer at Creusot works, was responsible for similar progress with those pieces of machinery which could not be rolled owing to their shape. The technique of iron metallurgy was thus acquiring a position in which it could meet the demands for cast and wrought iron which the developments in railways, naval construction and engineering were on the point of creating.

The manufacture of good-quality steel alone remained outside the general trend of progress. It was still dominated by an absolute empiricism, due partly to the fact that chemistry was in its earliest infancy, whereas mathematical sciences had already reached a high degree of development. Until Berthollet's time, at the end of the eighteenth century, the part played by carbon in steel was hardly suspected, whilst the action of phosphorus and sulphur was still unknown in 1846, when Leplay wrote a paper—a most judicious one from the economical standpoint—on metallurgy in Northern Europe.

Steel was solely used in trades which required only relatively-small quantities, namely, for making such things as tools and arms (6,000 tons were used in France in 1836). It was manufactured by two processes. There were the so-called "natural" classes of steel, such as those of Styria, which were obtained by refining in such a way as to leave carbon in the metal. The balls were squeezed, drawn in the shape of rods, and then redrawn, the piling and rolling operations being repeated once or twice according to the degree of refining required. The other method consisted of cementing with charcoal grades of iron as pure as possible, the best brands coming from Sweden, notably from Dannemora. They were drawn, piled and redrawn after cementing, or, for preference, they were melted in the crucible, which had then been known for about a century, and were cast into ingots.

The manufacture of the fine classes of steel had reached a high stage of perfection in Yorkshire. It was retarded in France by a wrong theory put forward by Réaumur in 1722, and afterwards stubbornly held by officialdom, that the grades of iron obtained from French mines, notably from the Berry province, were as a raw material equal to Swedish or Russian iron. Jars, one of the early founders of Creusot, had controverted that opinion in 1765, after a journey he had made to England, but he met with no support. It was only about 1830 that Yorkshire methods and Swedish iron were introduced in the Loire Department, and so reached France at the same time as the railways.

Since that date, France has been able to dispense with refined steel of foreign manufacture, and later even with foreign iron when progress in chemistry, the discoveries of Gruner on the action of phosphorus, and the turning to account of basic processes, allowed excellent refined classes of steel to be made from French ores, a result which Réaumur claimed as possible 150 years too soon.

How far had the important industries which were destined to become the principal customers of metallurgy during the last 100 years developed in 1836? The most important of these, the railways, were then in their infancy, but even then there was no lack of clear-sighted men who, to some extent, were able to foresee the gigantic extent to which they were destined to develop. The railway industry in its essential features, and with the magnificent prospects it was holding out to the world, may be said to have proceeded from an English colliery, and, to me, there is something pathetic in this gift to humanity of power and speed, for it is a gift which was made, as it were, by an unknown miner compelled to carry out, slowly, like a mole, his burrowing work underground. Thanks to him the inhabitants of even the most remote localities are now in a position to benefit by all the numerous improvements of civilization.

The rail, which reduces tractive resistance by nine-tenths, and allows of safe travel at the greatest possible speeds, had its origin in the quarry and the colliery. It was made of stone, trough-shaped to start with, but this was replaced later by a wood beam; in the eighteenth century a cast-iron straight-edge was used, and was replaced by a wrought-iron bar in 1808 thanks to Cort's puddling furnace and rolling mill.

Trevithick and Stephenson were also miners; they were bold enough to fit a steam cylinder and boiler upon a truck, the heavy and cumbersome children of the broken-winded, though marvellous, Newcomen engine. In France, it was the activity of the St. Etienne miners, coupled with the genius of Séguin, which was responsible for

the building of the railway line from St. Etienne to Lyons (1826–1830) at the same time as the noted Manchester to Liverpool line was being built in England.

Thanks to the inventive genius and the resoluteness of the two Stephensons, the Blenkinsop 4-HP. locomotive, weighing 5 tons, which in 1812 hauled slowly in a colliery near Leeds a small train of eight trucks forming a $3\frac{1}{2}$-ton load, developed in the course of 20 years, to the " Planet " whose main characteristic features are comparable to those of a modern locomotive.

A speed of 60 miles (100 kilometres) an hour was reached in 1835. Two years after the death of George Stephenson, the speed of the Crampton locomotives was nearly 90 miles (150 kilometres) an hour. The first locomotive built in our Creusot Works for the Paris-Versailles railway in 1838, with 535 square feet (50 square metres) of heating surface and weighing 15 tons, was for that age a strong and powerful engine. I may add that a few years later my firm supplied the Great Eastern Railway with sixteen locomotives. This event—perhaps unique in the annals of British railways—aroused great enthusiasm when my grandfather, then President of the " Corps Legislatif " in France, announced the news to the Assembly.

The daring of inventors would certainly not have been limited to the loads hauled and to the commercial speeds then reached, had not metallurgy proved inadequate to yield steels of the required quality, together with plates and forgings of suitable dimensions. The inadequacy of metallurgy here acted as a brake; different qualities and also larger quantities of rails, tires and boiler tubes were needed. When the eagerness of promoters, having capital at their disposal, urged the extension of railways at a rate which metallurgy was unable to follow, the prices of materials rose to extremely high levels; bankruptcies followed, and this also checked the laying of new lines.

In that same year 1836, steam navigation was also in its infancy, and it met with more competition from sail navigation than anything the railways had to suffer. Following predecessors of the past—such as Denis Papin in 1700, and Jouffroy d'Abbans in 1783—Fulton succeeded in 1807 in launching on the Hudson the first steamboat able to provide a regular service. The " Savannah," a sailing ship fitted with an auxiliary steam engine, crossed the Atlantic in 1818; the " Sirius," in 1838, was the first ship to cross it entirely under steam; it was followed very shortly after by Brunel's " Great Western." The first satisfactory trial of the screw for ship propulsion was made in 1835. In that same year France owned eighty-two steamships, compared to the five hundred owned by England. At about the same time metallurgy started an important new departure, the use

of steel in the building of ships' hulls. The "Great Britain," an iron, screw-propelled ship, was launched in 1843, and this venerable hulk was still being used as a coal depot in the Falklands Island at the time of the celebrated battle of October, 1914.

As in railway construction, the boldness of engineers in ship designing often outstripped existing facilities. In 1857, less than 20 years after the crossing of the "Sirius," Brunel built the "Great Eastern," a ship about 700 feet (211 metres) long, having a displacement of 32,000 tons, and 8,300 HP. engines, features which—apart from power—are characteristic of modern liners. About that time the first armoured frigates were built for the French Navy.

Turning now to the realm of mechanics, we find in the year 1836 that the use of the steam engine was already becoming widespread. The total power installed in France in 1838 amounted to 26,000 horse-power on land and to 12,000 horse-power in merchant ships. The largest units rarely exceeded 100 to 150 HP., but engines of 300 to 400 HP. were under construction for the Navy.

The first water-turbines were coming into use. In 1833, Fourneyron built near Gisors the first water-turbine revolving on a vertical shaft, whose efficiency reached 80 per cent. He erected in the same year a 60-HP. turbine at Saint Blasien, in the Black Forest. Poncelet had greatly improved the water-wheel by making the buckets curvilinear in shape, thus increasing its efficiency.

Electricity had not then emerged from the laboratory stage, but Faraday had established the laws of induction, and Ampère, in 1836, had discovered those governing the action of a magnetic field upon currents. This marked the first origin of generators and motors, which were destined to play such an important part in industry towards the end of the nineteenth century.

The large industrial undertakings, which may be said to constitute the backbone of modern civilization, were still in a rudimentary state in 1836, but the principles of industry were already established and were solely awaiting favourable circumstances for production to reach in every respect the development we marvel at to-day. Among these circumstances one of the most important was the development in the manufacture of iron and steel to enable products to be provided which were suitable for requirements both as regards quantity and quality.

In regard to quantity, output had, on an average, to be doubled every 20 years in order to meet increasing demands. This was achieved without increasing the number of works—the number was, on the contrary, decreased. Wood fuel had to be discarded, as sufficient could not be provided for rising consumption. The capacity of the furnaces and the power of the blowing engines were increased.

The consumption of ore was increasing so rapidly that the important iron-ore mines of Lorraine, Bilbao, and Sweden were the only ones capable of complying with the demand; hence the concentration of the French metallurgical works on the Lorraine ore basin, or on the Northern France collieries. Such a concentration was facilitated by the railways.

The refining hearth, a heavy consumer of charcoal and labour, gave place to the puddling furnace. The size of the puddling furnace remained small for many years. Thus, Creusot forge, built between 1861 and 1867, had for an annual output of 110,000 tons—a considerable figure for that time but by no means so to-day—one hundred and thirty puddling furnaces, thirty steam hammers, and forty-one rolling sets, including twenty-six finishing mills.

But Bessemer in 1856 had discovered his process, a process which made steel a metal for general use and not a precious metal as it had previously been. Martin was the first to succeed at Sireuil in melting steel on an open hearth, thanks to the Siemens recuperators. His apparatus came at the right time for utilizing all the old rails which then encumbered the railways, and all the available crop-ends. It allowed of a higher-quality steel than could be produced by the Bessemer process.

Then in 1878 there appeared a new invention of outstanding importance, namely the Thomas and Gilchrist basic process; this was first used with the Bessemer converter and extended in 1880 to the open-hearth furnace. This invention made it possible to utilize the large deposits of phosphoric iron ores which constituted the greater mass of the European ore reserves, and more especially of those existing in France. This meant a revolution in the realm of metallurgical raw materials and led to a large concentration of the steel industry in proximity to the Lorraine basin.

The Creusot forge based on the puddling furnace had not been completed when the first open-hearth furnace, having a capacity of 4 tons, was built in 1866. The first series of (acid) Bessemer converters dates from 1870; these were transformed into Thomas (basic) converters in 1879. It can be seen, therefore, that progress left no respite to those ironmasters who were determined to remain in the first rank. The rapid increase in consumption also gave them no relief.

The electric furnace, introduced in 1900, meant for the refined and special steel industry an advance similar to that afforded by the Bessemer converter in the manufacture of ordinary classes of steel. My firm collaborated with Héroult, the inventor of the furnace which is now the one most extensively used. Had it not been for the electric furnace, which is more economical than the crucible, especially in

countries poor in coal but having numerous waterfalls, the price of special steels would have been prohibitive for many purposes.

The production of molten steel in large masses at a comparatively low cost, coupled with the fact that internal defects, such as piping and blowholes, could be prevented, placed metallurgists, between the years 1880 to 1890, in a position to cast in steel parts complicated in shape, which until then had been made of cast iron. These steel castings met a long-felt need and their manufacture developed rapidly. In numerous instances, thermal treatment made the steel castings as safe as forgings.

Forging tools progressed on similar lines in regard to power, precision, and ease of action. Our Creusot works owned a 100-ton steam hammer in 1877; later the Bethlehem steelworks built one of 130 tons. But the forging press invented by Bessemer, Armstrong and Whitworth in about 1860, ultimately replaced the hammer for heavy forgings, the hammer, whose action is more violent, being retained for medium and small-sized forgings only. There are now in different countries several quick-acting forging presses of 15,000 tons, and even of 20,000 tons, capable of forging 250-ton ingots.

Developments in the rolling mill have been no less remarkable; the rolling mill is the machine for manufacturing practically all steel products in general use. The old two-high mill invented by Cort was replaced in 1861 by the three-high mill having a larger output, and in 1866 by the reversing two-high mill for the rolling of large sections.

The era of electricity commenced at the close of the nineteenth century and radically transformed the rolling mills. Whilst taking up little space and being easy to operate, motors are now built of considerable power, even exceeding 20,000 HP. in the case of some reversing mills, which is a step forward since the 50 HP. engines of 1836. Thanks to electricity, the mills have as accessories a whole series of automatic apparatus such as travellers, lifting platforms, skid rollers, shears, and so forth, which, whilst making for increased output, decrease labour costs. The rolling-mill sheds have been lengthened, and the bars rolled are longer. Outputs exceeding 1,000 tons per 8-hour shift are of frequent occurrence. In 1865 our Creusot works needed twenty-six rolling-mill sets, many of which were of the same sizes, to roll 100,000 tons per annum. In a modern steelworks of 1 million tons capacity, five graded sets suffice for producing all the usual bar sections. At the present time one specialized rolling-mill set (as, for example, for rolling rails or heavy plates) would by itself suffice to cover the needs of a large country if it were to work without stopping.

The rolling mill has extended its realm beyond bars and plates,

and its extension to the manufacture of weldless tubes is one of the important conquests of metallurgy. Of late years cold rolling has become a feature of the manufacture of thin sheets for galvanizing and for the tinplate trade, and has made available for stamping out sheets of remarkable ductility and perfection of surface, which are specially required for the manufacture of motor-car bodies. At the same time the cold working-up of steel has led to important investigations being made regarding the phenomena of the ageing of metals and of recrystallization.

The progress of metallurgy from the standpoint of quality has been quite as important as from that of quantity, if not more so, for, in regard to quality, work had to start absolutely *ab initio*. Science took its first steps under the auspices of analytical chemistry. The determination of carbon, phosphorus, sulphur and manganese brought out the influence which these elements had upon the quality of steel, and enabled methods of manufacture to be improved. It explains the failure of the attempts of the Réaumur school to manufacture refined steel starting from pig-iron, all of which was phosphoric to a greater or lesser degree. The discovery of the basic process, applied first to the converter and then to the open-hearth furnace, and the use by Pourcel of ferro-manganese were the most important achievements of the period under review.

The ideas held concerning these factors were, however, still in a rather confused state, for there were many doubts as to the real action of the elements associated with iron, either as impurities or as additions. Micrography and the application of the theory of alloys allowed a great step forward in the study of this action. Réaumur in 1722 had made an examination of the structure of steel under a microscope, and later in the eighteenth century armourers resorted to the superficial attack of steel by acids. The combination of the microscope and the reagent was employed by Sorby in 1867 and then by Martens in 1878.

Two of my collaborators, Osmond and Werth, took up these studies with much greater precision, thanks to photography, and in 1885 established the proof of the cellular and crystalline structure of steel and alloys. Steel appeared as an aggregate of pure iron and of compounds of iron and other elements likely to form solid solutions or to become dissociated under thermal treatment. The relations existing between the state of the constituents of steel and its mechanical properties, and the influence of variations of temperature upon the state of the constituents of steel formed the subject of researches by many other scientists, among whom Le Chatelier, Roozeboom and Roberts-Austen were most prominent. The new methods of metallographic investigation were completed by resorting

to oblique illumination, which has allowed the identification of certain inclusions otherwise invisible.

Progress in the realm of chemical analysis has brought about the titration of occluded gases, particularly oxygen both free and in combination with slag inclusions, and the analysis is carried out simultaneously with the microscopic examination. Our French laboratories are studying micro-analytical processes, their ambition being to investigate micro-segregation.

New testing machines introduced by M. Chévenard allow of the determination of the mechanical characteristics of a metal piece at exact points by tensile, bending and shearing tests on very small test specimens, 0·06 inch (1·5 mm.) in diameter. These methods of investigation prove most useful in autogenous welding technique.

Notwithstanding its apparent strength, steel is a structure whose stability is threatened by numerous factors, such as chemical agents, temperature and frequently-repeated stresses even when they are too low to give rise to permanent deformation. Corrosion-tests, mechanical tests in the hot state, and repeated torsion and bending tests supply data which are absolutely essential to-day concerning the behaviour of the metal under the influence of the factors referred to above.

X-ray examination, which is still in its infancy, permits of a considered opinion being given on the homogeneity of steel without damaging in any way the piece under examination, and also brings to light internal defects such as porosity, blow-holes and cracks provided their thickness is sufficient. This method of investigation is most useful for the verification of welds in plate-work such as boiler-work or high-pressure pipe-lines.

The spectro-graphic analysis of a beam of X-rays, after passing through a thin strip of steel or steel powder, enables one to penetrate into the molecular structure of the metal and to catch a glimpse of the mechanism governing the action of the different constituents. This method will be used to determine with certainty the limits of solid solutions, to study definite compounds and their crystalline system, and also to locate the various changes occurring in the solid state. It will at the same time throw light upon the phenomena of cold-working and re-crystallization. This exploring of the microscopic particles will procure data of the highest interest on special steels, on methods of forging and also on the heat-treatments which are most suitable for their different uses.

One of the main tasks of the steelmakers was to discover the causes of the defects which had long been their despair: these included piping, segregation, inclusions, blowholes, cracks, contraction cracks, splitting, etc.; as a result it became possible to

prevent them, or, at all events, to lessen their effect. The part played by the slag was also investigated. The laws of equilibrium between the metal bath and the slag in a steel furnace are now generally known, and are made use of to carry out a rapid purifying of the metal.

I will not undertake to review, even briefly, the different classes of special steel made available during the last 40 years. The number of binary, ternary and still more complex alloys of iron, carbon and various metals such as chromium, nickel, molybdenum, vanadium and tungsten, now reaches a very high figure, and the properties of all such alloys can be made to vary in innumerable ways by changes in the forging and thermal treatment processes to which they are subjected.

Generally speaking, when it is necessary for a piece to display mechanical characteristics uniform throughout its whole thickness, the steel alloy which will give the desired quality with the lowest percentages of noble metals, and therefore the steel alloy the lowest in cost, is the one whose critical rate of quenching is only slightly lower than the rate of cooling of the neutral plane of the piece during the process of quenching. In such conditions, one can be certain that annealing will give the piece throughout its entire thickness a sorbitic structure, which is the best possible one for bringing out the mechanical properties of the steel. Nickel is the best alloy metal for acting upon the critical range of quenching, since it increases hardness without making for brittleness, and allows of decreasing the percentage of carbon when an increase in hardness is not desired.

Among the most interesting alloys may be mentioned high-speed steels; also carbides, mainly tungsten carbide. These alloys have revolutionized the mechanical engineering industry by allowing of rapid machining, and thus increasing to an unsuspected degree the yield of labour and machine-tools—a result which is not without its disadvantages from the social point of view.

Other interesting alloys include stainless and semi-stainless classes of steel. The object governing the manufacture of semi-stainless steel has been to obtain at not too high an increased cost a steel capable of resisting, not indefinitely but longer than ordinary steel, the action of the air in the atmosphere of a works or the corroding action of water, chiefly sea-water. The use of very pure iron alone or with the addition of a proportion of copper or molybdenum has given interesting results in particular instances; much, however, remains to be done in order to protect steel of everyday manufacture, even when covered with a coat of paint, from corrosive action, that arch-enemy of all steel structures.

When the question of cost does not prohibit the use of noble metals

on a large scale as alloying elements, special steels can be obtained capable of completely resisting, not only normal atmospheric agents, but also a large number of reagents. The first really-rustless steel made was probably ferro-nickel containing 25 to 30 per cent. of nickel, and placed on the market in 1890 ; its manufacture in the open-hearth furnace was started in Creusot at about that date. The use of chromium, the addition of molybdenum and of titanium, the perfecting of thermal treatment and of pickling and the smoothing of the surfaces, all combined, permitted metallurgy to offer to the chemical industry a whole series of steels having notable mechanical properties, which could be worked up, machined and welded, and were at the same time capable of resisting indefinitely, or for long periods, the strongest chemical reagents. It is necessary, however, in each case to select a suitable class of steel for each corrosive substance. A universal solution of the problem is not yet possible.

Among the steels here referred to, the best known and the most commonly used is the one having an austenitic structure, containing 18 per cent. of chromium and 8 per cent. of nickel. The uses to which steels of this class have been put have led to important laboratory experiments being undertaken, owing to the instability of their structure when welded. This is shown by carbide formations at the boundary of the polyhedral crystals, such formations indicating a high degree of brittleness.

Metallurgy has been busy of late years in the study of a steel which shows satisfactory resistance both to mechanical stresses and to oxidation at high temperatures. Research on these lines is most interesting, particularly with reference to internal-combustion engines whose efficiency rises with the temperature of the cycle. For example, a nickel-chromium-molybdenum steel or a chromium-tungsten-vanadium steel has the same elongation at 930° F. (500° C.) under a stress of 13 tons per square inch (20 kilograms per square millimetre) as a mild ordinary steel under a stress of 3 tons per square inch (5 kilograms per square millimetre). Chromium steels containing a certain proportion of nickel, tungsten and vanadium, withstand temperatures of 1800° and even 2000° F. (1000° and 1100° C.).

The manganese steels for which Sir Robert Hadfield is responsible deserve special reference. Their cost is relatively low and they are specially suitable for the manufacture of pieces which have to resist wear and violent shocks, such as the component parts of crushing machines, and railway and tramway crossings. The extraordinary strength of these steels, whose elongation may be as high as 50 per cent., led to their being used for the making of helmets during the Great War, and many men owe their life to this remarkable alloy.

The progress in regard to quantity and quality made by metallurgy

has exercised a marked influence on the different branches of applied mechanics; it may even be said to have created several of them. The total length of the railways now covering the world's surface amounts to over 800,000 miles (1,300,000 kilometres). Although the construction of new lines has slowed down considerably owing to the competition of other means of transport, the world required 7 million tons of steel rails in 1929. In order to resist the stresses set up by modern fast and heavy trains, the weight per yard has been increased and the railhead has been subjected to a thermal treatment of quenching followed by annealing, the flange and web retaining their full strength. The switches and crossings are frequently made of quenched 12- to 14-per-cent. manganese steel, as resistance to wear and non-fragility render this class of steel most valuable for such a purpose. Locomotives have maintained the general features given them by Stephenson in 1830, but their power, weight and efficiency have all increased enormously.

Weldless steel tubes and plates, first of mild steel and then later of nickel steel, are able to withstand much higher pressures. From 8·5 lbs. per square inch (0·6 kilograms per square centimetre) in the case of the first tubular boiler built by Séguin, the pressure rose to 140 lbs. (10 kilograms) in 1870, to 210 lbs. (15 kilograms) in 1900, 280 lbs. (20 kilograms) in 1930, and locomotives with a pressure of 1,400 lbs. (100 kilograms) are being tested. Superheat, as well as double and even triple expansion, reduces steam consumption. The steam locomotive, although a hundred years old, is still able to contend with its younger competitors, the electric and diesel-electric locomotives and the autocars.

Axles, tires and, in certain instances, frames also, are of nickel or nickel-chromium steel. This helps to reduce weight whilst making for power and speed, combined with safety. Occasionally also frame plates are cast in one piece with the cylinders.

Railway-carriage bodies are built of metal sheets instead of wood, and there is a tendency to replace the mild-steel sheets now used by sheets of nickel-chromium-molybdenum steel whose elastic limit is almost double that of the former. Even austenitic steels have been used for this purpose in America. These are cold-rolled and their tensile strength reaches 82½ tons per square inch (130 kilograms per square millimetre). Combined with electric welding, they make for a 35-per-cent. reduction in weight.

Thanks to all these improvements, which are due in part to the steel-maker, 1,000-ton trains can now travel at a speed of over 80 miles (130 kilometres) per hour very much more safely than did the trains of 1840 weighing 100 tons and running at 25 miles (40 kilometres) per hour.

Whilst the railways are beholden to special-steel metallurgy for the considerable progress they have achieved, their great modern competitor, the motor-car, owes to that industry its very existence. In the case of the motor-car, lightness is a vital necessity. No motor-car can be conceived in which the crankshaft and gear wheels are not of special steel or the valves not of chromium or tungsten steel any more than it can be conceived without rubber tires. The casting in one piece of its complicated cylinder sets, and the manufacture of its light and safe springs and of the sheets forming the body, perfect in appearance and lending themselves to the most complicated shapes, are all masterpieces of metallurgical science.

Aviation has had to solve similar problems to those which confronted the motor-car industry, but in aviation difficulties of a special nature were superadded, since in its case lightness is of greater importance still. Recent military aviation engines weigh hardly 1·1 lbs. (500 grammes) per HP., as against 90 lbs. (40 kilograms) per HP. for a locomotive. The weight of the framework is also correspondingly light. Iron alloys are too heavy, and recourse must be had to those of aluminium and magnesium. Methods perfected in metallurgical laboratories have led to the manufacture of a whole series of light alloys having a specific gravity of 2 to 3 and a strength comparable to that of mild steel.

The improvements arrived at may be measured by calculating the vertical length of the longest round wire which can bear its own weight. This length amounts to:

2·5 miles (4,100 metres) for puddled iron,
3·3 ,, (5,400 ,,) ,, mild steel, and
6·2 ,, (10,000 ,,) ,, hard steel.

In the case of wire made of light aluminium and magnesium alloys, the length exceeds 10 miles (17,000 metres), and is about 15 miles (24,000 metres) for wire of special nickel-chromium-tungsten steel.

Bridges and structural work for railways and roads are also indebted to steel of high tensile strength, inasmuch as this steel, for an equal factor of safety, allows either longer spans or a decrease in weight. For a given tensile strength, special steels containing either nickel, a metal which has the disadvantage of being costly, or chromium and copper, or again silicon or manganese, have a greater elongation than carbon steels and are less liable to air-hardening. Thanks to the use of special steels the longest span in a cantilever bridge can be increased from 1,650 to close upon 2,300 feet (500 to 700 metres) and that of a suspension bridge from about 2,300 to over 4,900

feet (700 to 1,500 metres). These facts add to the great admiration one feels for the engineers who half a century ago built of mild steel, or iron, structures such as the Forth Bridge with its 1700 feet (519 metres) spans, and the Eiffel Tower.

In drawn steel wire the tensile strength reaches a maximum of about 115 tons per square inch (170 to 180 kilograms per square millimetre). Cables made of such wire allow of colliery winding-depths much above 3,280 feet (1,000 metres). At a winding depth of 3,940 feet (1,200 metres) a round cable of 115-ton steel can safely lift 14 tons per square inch (22 kilograms per square millimetre), as against 1 ton per square inch (1·6 kilogram per square millimetre) in the case of a 38-ton steel cable. The use of hoisting cages built of nickel-steel or of a light alloy will still further increase possible winding depths.

The part played by internal-combustion engines and turbines in the production of energy is now known to everyone. They owe their existence to the progress made by steel metallurgy in the manufacture of castings of steel and cast-iron alloys, and in the production of special steels for shafts, turbine-blades and valves. High superheat, to which is due the high efficiency of steam turbines, was rendered possible by chromium-nickel-molybdenum and chromium tungsten-vanadium steels, the only steels that can withstand high temperatures. It is quite possible that these same classes of steel will soon have to be used in the construction of steam boilers.

The gas-turbine problem depends almost entirely upon metallurgy for a successful issue.

Development in high-power turbines, forming part of hydro-electric generating stations equipped with flywheel alternators, is due mainly to the skill displayed in the steel foundry; in this instance any internal defect has to be guarded against owing to the grave risks incurred in the event of the machine racing. The manufacture of heavy rotors for alternating machines coupled to steam turbines, with massive shapes deeply hollowed out near the centre, is for the same reason one of the most intricate jobs which a steelmaker and a smith have to undertake.

One of the special conditions to be observed in regard to turbine shafts is to carry out their manufacture in such a way that there be no possibility of their undergoing deformation at their actual working-temperatures. This means that no residual stress should remain in them following thermal treatment. In France, MM. Mesnager and Jouguet discovered methods of arriving at the residual stresses in question; the methods are, however, costly and take time. My firm has perfected an approximate method, styled the "prism method," which allows in every case of determining the type of treatment

required, and which consists of measuring the deformation of a test prism in the course of machining.

In large distribution systems of 220,000 volts and over which extend over whole countries, as in France, capacity effects are such that it is advisable to instal three-phase generators of very great power. 150,000-kilowatt turbo-alternators have been put down. The same tendency is shown in regard to water-turbines. Risks increase to such an extent in the making of the flywheels for the latter that one prefers to build up the flywheel from steel sheets carefully prepared rather than to depend on a steel casting.

The marvellous impetus arising out of the inventive genius of Ampère and Faraday would have been greatly hampered had not metallurgists placed at the disposal of electricians silicon-steel sheets of low hysteresis and high resistivity, without which the cool running of high-power dynamos and alternators would have been impossible and the efficiency of these machines extremely low. I am pleased to mention here that silicon steel is another of Sir Robert Hadfield's alloy inventions. High voltages require to be transformed up and down by several steps, hence the efficiency of transformers becomes a question of primary importance. By regulating composition and rolling and thermal treatment, iron metallurgy has cut down cyclic losses, having in a very few years succeeded in reducing the same in the proportion of 3 to 1. One is bound, however, to add that the electrician has largely paid back the metallurgist by having completely transformed the steelworks in the course of the last 30 years.

The shipbuilding industry may be said to have been the favourite customer of metallurgy and many most important feats have been accomplished on its behalf. Whilst cargo and river boats only find it necessary to take from the steel industry large quantities of low-priced plates and sections for hull construction, and of castings and forgings for diesel engines, great liners, on the other hand, have to meet a cut-throat competition in the matter of comfort, speed and safety, and have, therefore, been customers much more difficult to please. Here, in order to lighten the hulls of liners, the tendency is to use plates showing 32 and even 38 tons per square inch (50 and 60 kilos. per square millimetre) tensile strength, and riveting is being discarded in favour of welding. As the addition of carbon is not suitable for hardening iron when welding is employed, some other element, such as nickel, has to be used.

Several parts entering into the building of the hull, such as the stem, sternpost and propeller brackets, are masterpieces of the steel foundry. The sternpost of S.S. *Normandie*, manufactured by the Skoda works in two pieces, weighs 102 tons; the outside propeller

brackets weigh 75 tons each and the inner ones 97 tons each. Such pieces, complicated in design and containing parts which vary greatly in thickness, have, of course, to be sound throughout.

For warships, the question of weight has nearly the same importance as for the motor-car or the aeroplane. The installation of 100,000 HP. steam turbines with numerous other machines inside a 2,500-ton ship, or of 6,000 HP. diesel engines with dynamos, accumulators and electric motors inside a submarine is no easy job ; but the duel between armour and the armour-piercing shell is particularly dramatic. Every new batch of plates has to be able to resist the best shell available in the Navy, and then a shell is looked for which can pierce the best plate ! Up to now each side has been successful in turn, but this striving after an ever-receding end is heart-breaking at times !

It is most satisfactory to note that all this unending work arising out of military requirements has brought about results which have their uses in peace. The researches made with a view to lighten submarine and aeroplane engines have led to improvements in agricultural tractors, motor lorries and autocars, whilst the experience gained in the manufacture of armour plates has been of use in the building of steam accumulators and of containers for the hydrogenation of coal and oils, and in other branches of the chemical industry.

In concluding this much too lengthy Lecture of mine, I wish to emphasize the solidarity which binds together every branch of human activity, the intermediary being the key industry " Metallurgy of Iron." Iron may be termed the universal material, by reason of the most remarkable facility with which it adapts itself, alone or in conjunction with other metals, to all possible uses, from a bar in reinforced concrete, to a watch spring or an artificial tooth. The laboratories of works, such as Creusot works and similar English steelworks, may be likened to the meeting place of a town, where the representatives of every individual science communicate to one another the improvements each one has carried out. This is also, and on a wider scale still, the role of important Institutions of engineers such as your own. Your Institution has always fulfilled its mission in the past, fully conscious of its high calling ; my most sincere and earnest wish is to see its radiance and influence extend farther and farther as the years go by.

The Meeting concluded with a vote of thanks proposed by Sir Robert Hadfield, Bart., and seconded by Mr. S. B. Donkin.

EXTRA MEETING.

16 June, 1936.

Mr. JOHN DUNCAN WATSON, President, in the Chair.

SPECIAL LECTURE ON

"Water-Power in Brazil, with Special Reference to the São Paulo Development."

By ASA WHITE KENNEY BILLINGS, M. Inst. C.E.

MR. PRESIDENT, Members of The Institution, Ladies and Gentlemen, I wish first of all to acknowledge the honour of being asked to speak on Water-Power in Brazil, and to thank the Council for their invitation, which was a result of Sir Richard Redmayne's visit to South America. I should like to mention here that in my opinion his visit and that of Lord Macmillan have been of considerable benefit. The esteem in which the British nation is held in all Latin American countries, including Brazil, is proverbial, but the intellectual relations and exchanges have not been as close as those with the Latin races in Europe, and so such visits help to make up for this lack of frequent contact. It was also the first time that the President of this Institution has visited South America in an official capacity; hence his visit was an especial satisfaction to all our members there.

The subject of my lecture may interest you because in no other country are physical conditions so favourable for the development of hydro-electric power on a large scale, and because, up to the present time, the principal factor in this development in the most important zone has been a Canadian company, the Brazilian Traction Light & Power Company, Limited. This company has invested over £75,000,000 in several of the public services of Rio de Janeiro, São Paulo, Santos and the surrounding region; these include water-power installations of 540,000 horse-power maximum capacity, as well as tramway, omnibus, gas, telephone and other services.

Brazil occupies more than half the area and contains more than half the population of South America; its area is about 3,200,000 square miles—equivalent to three times that of Argentina or to thirty-six times that of Great Britain; its population is over 45,000,000, or three-and-one-half times that of Argentina, and about equal to that of Great Britain. When Brazil is mentioned the

average person immediately thinks of the Amazon and of the country tributary to it; the real Brazil is very different. The enormous region of the Amazon has no appreciable water-power or population to be served; much of it is very sparsely settled, or inhabited only by Indians; its climatic conditions will probably never permit industrial development. In Southern Brazil, however, the climate and topographic conditions favour the development of a great industrial population and of an abundant supply of hydro-electric power.

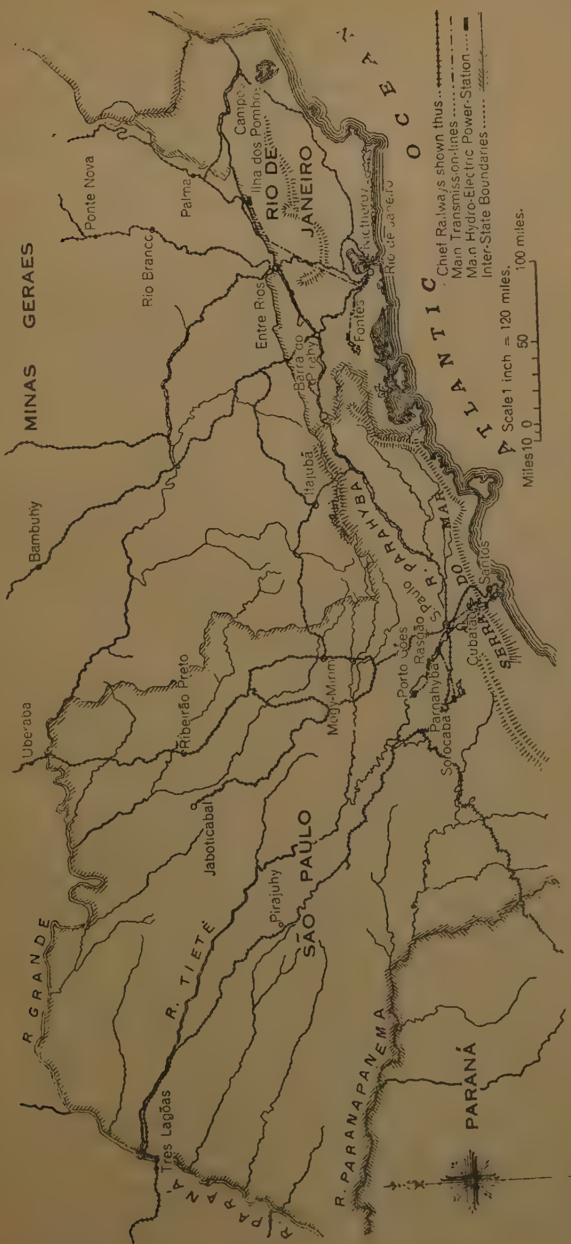
It is usual to publish statistics of the potential water-power of each country. The estimates generally given for Brazil range from 15,000,000 to 60,000,000 horse-power. It would seem that for countries like Brazil much of this statistical effort is wasted, as the visible waterfalls thus catalogued are secondary in economic importance to the invisible water powers obtainable by diversion. In addition, there is the difficulty of defining what really constitutes potential water power and for what period of the year it must be available to be counted, whilst, in the continents of South America and Africa which have the greatest potential resources, the lack of exploration or of observations of flow and head make such figures mere guesses. Only those powers which can be developed commercially within a reasonable period and are close enough to the centres of population to command a market, are worth study and tabulation.

Nearly all the great waterfalls of Brazil are relatively inaccessible and it seems certain that they are only of negligible importance in the economic future of the country; in fact, by obstructing navigation on the large rivers, they impede, rather than facilitate, the development of the region. All suffer the serious disadvantage of very high back-water in the flood season in the long narrow canyons formed by erosion by the fall itself. Thus the waterfall at Guayra is reduced from 380 to 263 feet, or by 117 feet, that at Iguassú from 230 to 125 feet, or by 105 feet, and that at Paulo Affonso from 262 to 170 feet, or by 92 feet. Others become mere rapids in the flood season.

The first of these falls—the Guayra fall—at the intersection of the Paraguayan boundary with the Paraná river, is the largest fall in the world. Here the Paraná has a minimum flow of over 400,000 cusecs, twice that of the Niagara river, while in the flood season it exceeds 2,700,000 cusecs, although the head, as already stated, diminishes under these conditions to 263 feet. The estimates of this one power vary from 12,000,000 to 20,000,000 horse-power; yet this, in all probability, will never be developed commercially for the reasons given. Curiously enough the power for the adjacent town and railroad shops is produced entirely by steam, using wood fuel.

Let us turn our attention to that portion of Brazil lying east of

Fig. 1.



General Map.

the Paraná river and between parallels of latitude 20 and 25 degrees south (*Fig. 1*). In this region the cooler and healthful climate favours the development of large cities and towns with their industries, while natural conditions facilitate extraordinarily the hydro-electric installations which serve and stimulate them. Rio de Janeiro has a population of 1,700,000; São Paulo, primarily industrial, has 1,150,000.

The development of this region in recent years has been stimulated especially by coffee cultivation; in the state of São Paulo alone there are 1,300,000,000 coffee trees which produce a third of the world's coffee. Over-emphasis on this one crop has its disadvantages and in the past few years Brazil has burned nearly 37,000,000 bags or 2,200,000 tons of coffee, in endeavouring to maintain prices; now other cultures, such as cotton and citrus fruits, are becoming important and more attention is being paid to manufacture and industry.

In this region hydro-electric plants, depending on their size and extent of regulation of flow, cost roughly the same as thermal power-plants; the expenditure on fuel for thermal power is, however, heavy. Oil fuel costs at present about 44*s*. per ton and the cost of coal is in proportion. It would seem, therefore, that thermal plants have no economic part in serving this industrial region of Brazil. In fact the high cost of fuel will result later in increasing the use of hydro-electric energy for thermal, as well as for electro-chemical purposes, and in certain cases the use of electric boilers will be advantageous. It may be remarked that Canada last year used in electric boilers 6,244,000,000 kilowatt-hours, thus avoiding the importation of 1,200,000 tons of coal.

He who travels by sea along the coast of this part of Brazil sees, for several hundred miles, what appears to be a mountain range, the Serra do Mar (*Fig. 2*). Geologically it is a fault or series of faults, as it forms the edge of a great plateau, about 2,500 feet above sea level, in the archaic or pre-Cambrian formation from which all the overlying strata have disappeared. The granite, gneiss and other rocks forming the edge of the plateau are decomposed only superficially and form an exceptionally impermeable soil, consisting of a natural mixture of sand and clay; in general the plateau is not eroded deeply and the river valleys are shallow.

In past ages the nearly level lower plain of denudation rose slowly, breaking along these faults parallel with the present coast. The rise, with the accompanying tilting toward the interior was, however, rapid enough, geologically speaking, to prevent the rivers which drained the plain from keeping pace with the rise by cutting down their courses to the ocean. The result is that, with few exceptions,


R. PARAHYTINGA
R. TIETÊ
Mogy das Cruzes
ATLANTIC OCEAN
R. ITAPANHAHU
Alto da Serra
Ribeirão Pires
Rio das Pedras Reservoir
Santos
S. Vicente
SERRA DO MAR
São Caetano
SÃO PAULO
Santo Amaro
R. GRANDE
Rio Grande Reservoir
R. PINHEIROS
Guarapiranga Reservoir
R. M'BOY GUASSÚ
R. TIETÊ
Parnahyba
Rasgão
R. S. LOURENÇO
R. JUQUIA
Railway lines shown thus
Transmission lines
Scale: 1 inch = 12 miles.
Mile 0 5 10 miles

Fig. 2.

the present rivers of this region follow the gentle slope of the plateau inland until they join the Paraná river; they then flow south-west and only join the ocean at the river Plate, more than 2,500 miles from their sources.

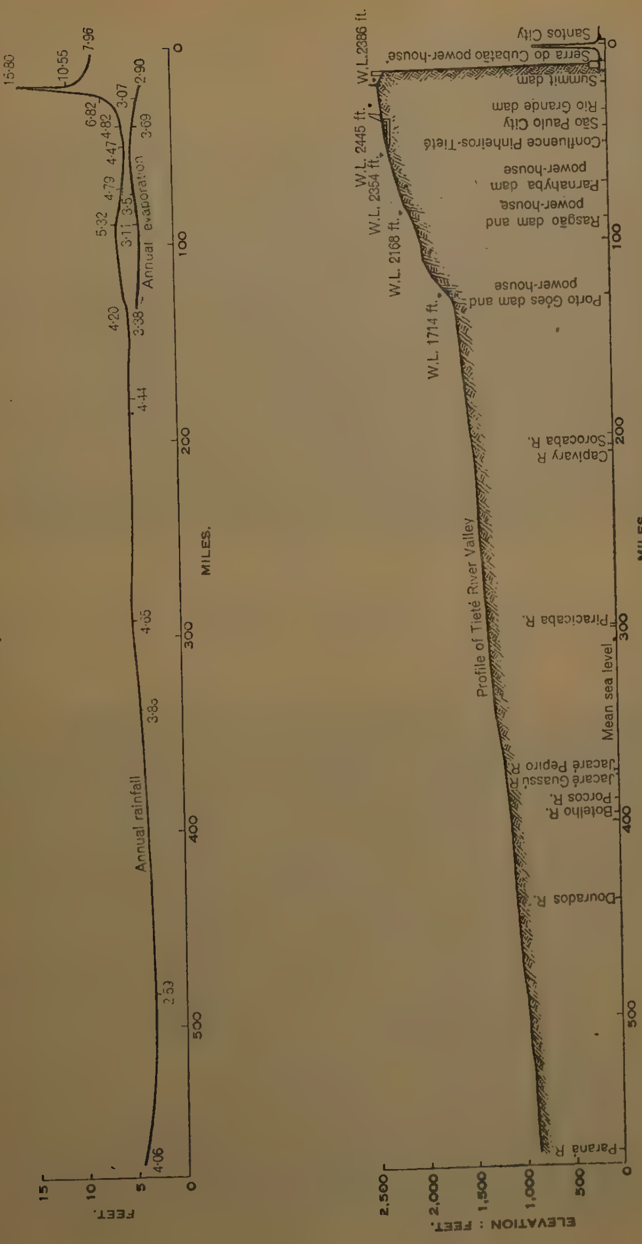
The slope of these rivers in their upper courses (*Figs. 3*) is very small, with a starting fall of about 1 in 3,000; hence the rains which fall on the plateau and which are especially heavy at its very edge, form sluggish rivers which flow away from the coast. In their lower courses, the rivers cross beds of harder rock, often basalt, and the resulting rapids and cascades, including most of the falls of Southern Brazil, have cut in these beds long canyons which keep on advancing imperceptibly upstream.

This extensive plateau, with its one or more abrupt steps gently sloping towards the interior, has very heavy rainfall; it receives this from three main types of disturbances—the active southern currents, originating frequently from the ocean, which underrun the continental air and are augmented by relief; the north to north-west monsoons, sometimes caused by mere convergence of continental currents in an intense convective medium or by their underrunning colder upper air; and the local convective thunderstorms, irregularly distributed. The first type includes also the aspiration of maritime air by the continental low-pressure areas.

Along the narrow hot coastal plain the annual rainfall is from 72 to 96 inches and along the crest of the plateau from 180 to 240 inches; it decreases inland to about 48 to 60 inches over most of the plateau. In the vicinity of the installation to which special attention will be given here, namely, the Serra do Cubatão plant near São Paulo, the rainfall at the crest averages 190 inches per year, and in wet years amounts to 270 inches. In wet months the rainfall reaches 52 inches, whilst the maximum observed in one day is 14 inches, and the heaviest downpour observed since this work started is 7½ inches in 1 hour and 20 minutes. The effect of such rains on the very steep slope of this fault zone has to be seen to be appreciated.

These amounts of rainfall, although from eight to eleven times the average at Greenwich, are not as great as those observed in some other parts of the world. For example, the Ghats in India have a rainfall of 360 inches in the several weeks of the monsoon season; Cherrapunji, on the Brahmaputra river, records a maximum of 560 inches in a year, with an annual average of 434 inches and a maximum in one day of 36·4 inches; in the hurricanes of the Caribbean Sea and in the typhoons of the Pacific Ocean rainfalls of 24 to 30 inches or more in a day are sometimes recorded. But the combination of heavy average rainfall, high head, favourable topography, sparse

Figs. 3.



RAINFALL, EVAPORATION AND PROFILE OF TIETÉ RIVER VALLEY AND SERRA DO CUBATÃO POWER DEVELOPMENT.

settlement of reservoir sites and proximity to a large and rapidly-developing market and to the sea, as in this part of Brazil, is rare.

Anyone who takes the trouble to examine closely the maps of São Paulo published by the State over 30 years ago, can see from the contour lines that a dam 90 feet high on the Tieté river a few miles below the City of São Paulo would back up this river, flooding a large part of the city as the water rose to form a lake of 106 square miles in area and of 2,000,000 acre-feet in volume at an elevation 2,408 feet above sea level. When this elevation was reached, the impounded waters would commence to flow in the other direction to the ocean over the lowest depression in the almost imperceptible divide near the edge of the plateau. This flow could thus be made use of in the drop of 2,380 feet to the coastal plain.

It would be impracticable, however, to build such a lake, for it would flood most of the city and destroy the market for power. The obvious alternative is to go upstream on the main river and on certain of its affluents, select dam sites and build reservoirs which can be interconnected and from which the water can be brought, as needed, to the edge of the plateau and to the power station at its base.

This simple idea is the basis of the Serra project; yet, strangely enough, it remained ignored and unstudied until about 12 years ago, when the first step was authorized by Federal decree in March, 1925, and by State law in December, 1925. In its original form the project consisted of a main reservoir on the affluent called the Rio Grande, which, at a maximum height of 2,421 feet above sea level, had an area of 26 square miles and a volume of 311,000 acre-feet. This was to be supplemented by twelve smaller reservoirs to be built successively as needed, and the surplus flow of each, after discharging downstream the " normal flow of the dry season " was to be diverted by canals and tunnels to the pipe-lines and power-house suitably located at the base of the Serra opposite the port of Santos. *Figs. 4* and *5* show the present power-house, pipe-lines and transmission lines, and the surge-tank 2,450 feet above the tailrace.

Further study during the initial construction period showed that it was advantageous to raise the level of the main reservoir by 22 feet, thus increasing the area to 44 square miles and the useful volume to 835,000 acre-feet, equivalent to 1,700,000,000 kilowatt-hours reserve storage. The other reservoirs were only slightly modified but the canals and tunnels connecting them to the main reservoir were eliminated; in their stead one important canal was projected by which not only the previously available surplus flow of the affluents, but also the floods and surplus flow of the Tieté itself could be diverted to the main reservoir and to the power-station by low-head pumping, through a difference in level of from 40 to 100 feet.

Fig. 4.

SERRA DO CUBATÃO, POWER-PLANT PIPE-LINES, AND TRANSMISSION-LINE.

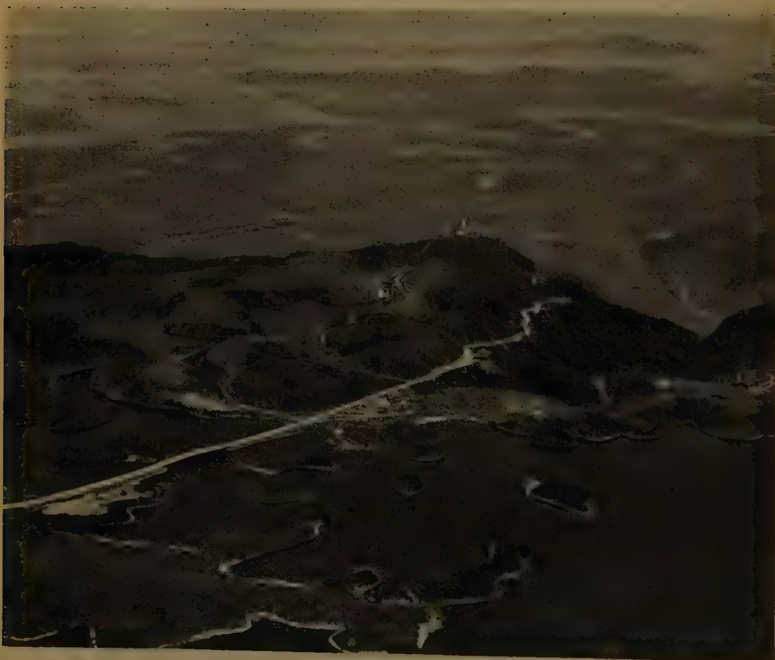
Fig. 5.

UPPER END OF PIPE LINES, SURGE-TANK, AND SIPHON.

Fig. 6.

RIO GRANDE DAM, RESERVOIR, AND CANAL.

Fig. 7.



RIO DAS PEDRAS RESERVOIR, TUNNEL-MOUTH, AND SURGE-TANK

This change seems complicated but is, in fact, a definite simplification and improvement, and the further reduction of the Tieté floods thus made possible is a great benefit. It should be noted that there are no irrigation demands downstream. The later reservoirs now authorized will have a total storage of 1,270,000 acre-feet, equivalent to 2,600,000,000 kilowatt-hours additional reserve storage. This, however, is not sufficient to regulate the flow of all these streams over at least the 11-year cycle which is characteristic of the climate of this region.

The first large reservoir, the Rio Grande (*Fig. 2*, p. 681), with the small reservoir leading the flow to the crest of the Serra, gives a regulated flow sufficient for 246,000 horse-power at 60-per-cent. load-factor. The diversion to the coast of the flow of the second large reservoir (the Guarapiranga built in 1908 to provide partial regulation for one power plant on the Tieté river) gives 140,000 horse-power more; this flow is obtained by discharging the water, when convenient, from the reservoir into the rectified channel of the Rio Grande river and pumping it through 25 to 85 feet difference of level into the Rio Grande reservoir. Further supplies up to 550,000 horse-power as an economic limit can be obtained by pumping the flow of the Tieté river up the rectified channel of the Pinheiros river through one step of 15 feet and then the one just mentioned of 25 to 85 feet into the main reservoir. Even this does not complete the project; the ultimate capacity and the order of development of the further water supplies will be determined by the economic conditions of the future which cannot be predicted accurately at the present time.

It may be said that each square mile of the drainage area provides water for from 600 to 2,500 horse-power, averaging 1,000 horse-power at 60-per-cent. load-factor; expressed otherwise, each mile of ocean front, measured along the edge of the plateau of the tributary drainage area, gives water sufficient for 10,000 horse-power. Evidently this development must be considered, not as a single installation but as a flexible programme of many steps, each to be carried out when needed; this will take care of the growth of load during perhaps the next 20 to 30 years. Up to the present over £5,000,000 have been spent; the ultimate total will depend upon the capacity which is found to be justified.

At the two main pumping stations mentioned, it is intended to instal successive units, up to a capacity of 5,000 cusecs or more, and to make them reversible so that they can operate as generating units at a minute's notice; they can thus assist in carrying the load at times of emergency. It is expected later to extend these periods of reversed operation, so as to assist regularly in carrying the day load in the dry season, repumping at night the water thus discharged

in addition to pumping the flow normally available. Evidently such a course, with its waste of energy, could only be adopted in a hydro-electric system without thermal production, where the average hydraulic efficiency is less important than the reduction of capital charge on the plant required to carry the peak loads and to provide sufficient reserve.

For the range of from 25 to 85 feet operating head mentioned previously, preparations are being made for two-speed units of 12,900 HP. (12,500 KVA.) at either 112·5 or 150 revolutions per minute, each being capable under maximum head of pumping 1,100 cusecs or of generating 12,000 HP. by discharging 1,450 cusecs. The first unit will, however, be of only half this size, provision being made for a maximum of six of the larger units, if required. The ultimate capacity of the plant in that case would be 83,700 horse-power when pumping or 56,550 kilowatts when generating. It is intended also, to a limited extent, to continue pumping flood waters even when the main reservoir is full, in order to assist in the reduction and control of the floods at the Tieté.

The benefits to the city of São Paulo due to this partial flood control are important. Usually in Brazil the great expense of sufficient flood protection cannot be justified by the resulting gain in value of the lands thus freed from flooding. In this particular case, however, the city is hampered in its development at present by the lack of large areas suitable for industrial sites, because the valley lands, the only area otherwise suitable, are covered with water during from 2 to 6 weeks of nearly every year. This will be remedied in part by the Serra development.

The first large reservoir, the Rio Grande, has been built only slowly and is being completed this year; the dams are made by the hydraulic fill process, the more important ones having plain concrete core-walls extending to the underlying rock for protection against burrowing animals and ants. More than 10,000,000 cubic yards of earthwork have been placed in the various dams, dikes, road diversions, etc., for the first large reservoir, whilst the second step, the canals for rectification of the Pinheiros river, involves about 15,000,000 cubic yards more. This is being moved cheaply by centrifugal dredging or by hydraulic sluicing, using electric power. *Fig. 6* (facing p. 685) shows the Rio Grande dam and the beginning of the main canal which will be over 200 feet wide (four times the width of the initial cut), 25 feet deep, and 16 miles long. Beyond is the main regulating reservoir and on the horizon the clouds marking the edge of the plateau.

Artificial sand is used exclusively in all the concrete work, with excellent results; the natural sands of this region are somewhat

unreliable, unless carefully washed, due to admixture of clay and organic matter. These methods of using artificial sand were developed by us 25 years ago and have been applied with uniform success in Spain, Mexico and Brazil in placing over 1,000,000 cubic yards of concrete.

The preliminary surveys, before construction was started, required many months of tedious exploration, reconnaissance lines and accurate levelling, on which the subsequent work could be safely based. Such surveys are hampered greatly by the dense undergrowth, which compels the cutting of thousands of miles of trails, veritable tunnels in the mat of vegetation, and slows progress down to a fraction of the ordinary rate in the open. For example, in the not specially-difficult topographic survey of the first large reservoir, over 2,500 miles of such trails had to be cut.

The selection of the best route to and down the face of the Serra required careful study ; this route, curiously enough, was found to run directly beside the automobile road from São Paulo to Santos. The small stream called the Rio das Pedras, the bed of which is used as a natural channel, had been rejected by others in earlier searches for power sites. Its minimum flow was only 4 cusecs while its maximum observed flood exceeded 5,000 cusecs. It is true that by itself this stream was not worth developing, but it is an essential link in the whole scheme.

At the regulating dam between the two reservoirs, where the flow regulated by the larger is discharged as needed into the smaller one leading to the Serra, three propeller-type turbines, each of 7,000 HP. capacity at full head, will be installed to utilize the variable head of 60 feet or less, which would be otherwise wasted.

A concrete dam, 85 feet high, was built to close the gorge of this small stream at the edge of the Serra, forming a lake of less than 3 square miles, which contributes, however, water sufficient for about 28,000 horse-power and serves as a natural forebay for the main power-plant.

From an arm of the smaller lake the water is passed through a tunnel 10 feet 8 inches in diameter and 1,230 feet long, and a steel siphon and connections 380 feet long, to a surge-tank on the summit of the steep spur descending to tide water on the coastal plain ; this surge-tank serves four pipe-lines, each connected to an individual generating unit. *Fig. 7* (facing p. 685) shows the smaller lake, the mouth of the tunnel, the surge-tank and the coastal plain. A second tunnel alongside is partially constructed which, with its siphon and surge-tank, will serve the next four pipe-lines and units ; provision is also made for a third tunnel to serve four units more, if and when the demand arises. Thus the first two units of 40,000 HP.

rating each, the third of 60,000 HP. rating and up to nine more of considerably larger size can be installed and supplied with water at 60-per-cent. load-factor; the present system load-factor is over 60 per cent. (63 per cent. correcting for growth) and will rise gradually to more than 70 per cent. in the later stages.

The concrete anchor blocks number fifteen and weigh from 1,000 to 3,200 tons each; the total volume of concrete required for them as well as for the piers, etc., for three pipe-lines amounts to 25,000 cubic yards. Proportionate amounts will be added for each future pipe-line. The pipe-line slope is 5,100 feet long and 120 feet wide, including the permanent inclined railways and the main drainage ditches; the average grade is 51½ per cent., with a maximum grade of 86½ per cent. The heaviest pieces of pipe do not exceed 13 tons, and all transport of these is handled by cable inclines located temporarily beside each pipe-line as it is being erected.

The cost of a pipe-line, relatively insignificant in a low-head plant, is in a high-head plant about equal to that of the corresponding turbine, generator, transformers, auxiliaries and switching apparatus combined. Moreover, an accident to a high-head pipe-line may be costly, endanger life and property and disrupt important public services for considerable periods. In this case several pipe-lines are required, each costing (delivered but not erected) from £50,000 to £60,000, and these are unavoidably located on a steep slope composed wholly of highly-fractured and partially-decomposed rock. Great precautions accordingly have to be taken to prevent slides, which might be caused by injudicious excavation, depositing of material on the slopes, entry of rain water into the formation, or even by a possible leak from a pipe-line. The importance, under such conditions, of intelligent pipe-line design and execution cannot be over-emphasized.

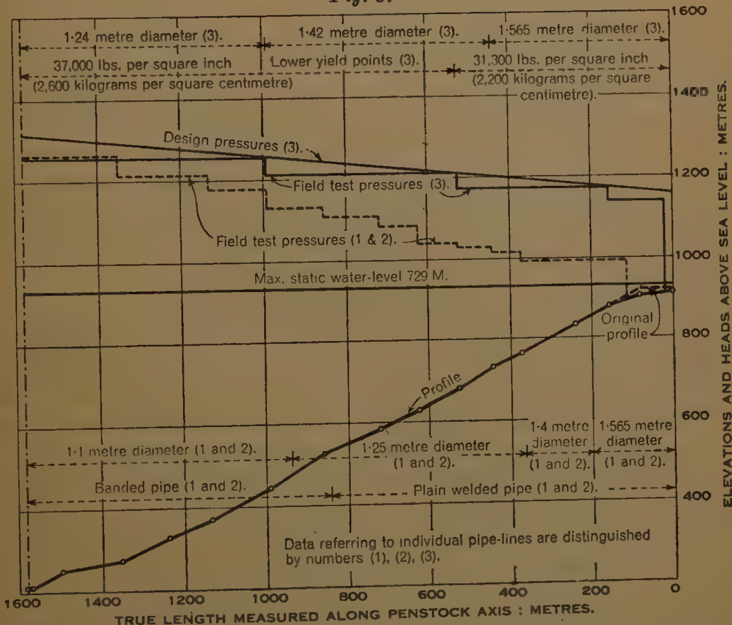
The first two pipe-lines, completed in 1926, were designed along nearly conventional lines—that is, by assuming a slow surge due to extreme governor action of 12½ per cent. in excess of the static pressure at each point, and then proportioning the thickness for a "factor of safety," referred to the ultimate strength of the steel and based on circumferential stress only, of 3·9 in the plain pipe and of 3·4 and 3·1 in the banded pipe. Under these conditions the pipe-lines had 700 tons each of plain pipe and 954 and 894 tons respectively of banded pipe.

They were tested in ten sections after erection to pressures ranging from 1,502 lbs. per square inch, or 146·5 per cent. of static pressure, at the lower end, to 350 per cent. of static pressure near the upper end. In subsequent operation they have been subjected with satisfactory results not only to the usual conditions of

heavy operating service, but also to many special tests during which heavy surges were produced intentionally and recorded.

The third pipe-line was designed differently without banding, using in the lower part a special steel of appreciably higher strength. The design took into account the stresses in all three directions, and was based on not exceeding 80 per cent. of the lower yield-point under the extreme emergency conditions which were assumed for purposes of calculation to be equivalent to the instantaneous stopping of one-half the rated overload flow. In comparison with the earlier pipe-lines, the conventional "factor of safety" in the lower part is about the

Fig. 8.



same (3·1), but that in the higher part is much larger, being 12·0 at the upper end; the pipe-line weighed 1,747 tons. Its unit cost per horse-power was approximately half that of the earlier pipe-lines, but such comparisons are misleading due to the differences in diameters and the marked fluctuations in commercial conditions, freights, rates of duties, and especially in exchange. The profiles of the three pipe-lines described, with the heads adopted for design and test, are given in *Fig. 8.*

Although these pipe-lines and their accessories have given excellent results, further experience has led to the conviction that the bases of all conventional designs for such service are so far from the truth that

they are wholly unsatisfactory. A long and intensive study has been carried on, a preliminary report of which has been published recently.[1] This gives what may be called a progress report, covering conventional and newer methods of design, the meagre data obtainable regarding accidents and extreme surges in practical operation, the importance of accidental conditions, the mathematical theory of surges, certain graphical methods, the importance of the partial and diffuse reflections, usually ignored, of pressure waves, tests of full-sized pipes, and the hydraulic and mechanical details of design. These studies are being continued, and the complexity of the subject makes it probable that they will be continued as long as there are more pipe-lines to build.

One might attack these difficult problems more leisurely, but the growth of load hinders the investigations one would like to make. It is not possible to avoid making a compromise between the conventional but inadmissible design and the ideal but as yet unattainable one. It is safe to say that no two of the several pipe-lines to be built in future will be exactly alike; if they are, it will be a sign that progress has ceased, rather than that the ideal has been reached. Meanwhile it is necessary to strike knowingly a balance between economy and safety, instead of blindly letting a conventional "factor of safety" (which in this case is a factor of ignorance) take care of the unknown factors. This is at present almost the only way to improve practical pipe-line design, because the small total number of pipe-lines built each year, the wide difference in conditions of the various installations, and the secrecy surrounding troubles or accidents, prevent the steady improvement of design on the basis of accumulated experience, which is the normal course in the design of other engineering structures.

It should be noted that the frictional loss in such pipe-lines is an important factor in both the design and operation of the plant; initially the corresponding loss in capacity of the turbine is balanced against the interest on the capital required to diminish this frictional loss. In a warm country with soft moorland waters, a slime accumulates on the inner surface of the steel at a rate which increases the friction by about 10 per cent. each year; that is, the maximum capacity of each unit diminishes roughly 1 per cent. each year. Anti-fouling paints, containing usually mercury or copper compounds, diminish, but do not entirely prevent this growth.

The internal protection of the various pipe-lines is accordingly important, but no trouble is experienced with external corrosion; a bituminous paint of good quality affords ample protection. For the

[1] "Symposium on Water Hammer," Am. Soc. Mech. E. and Am. Soc. C.E., 1933.

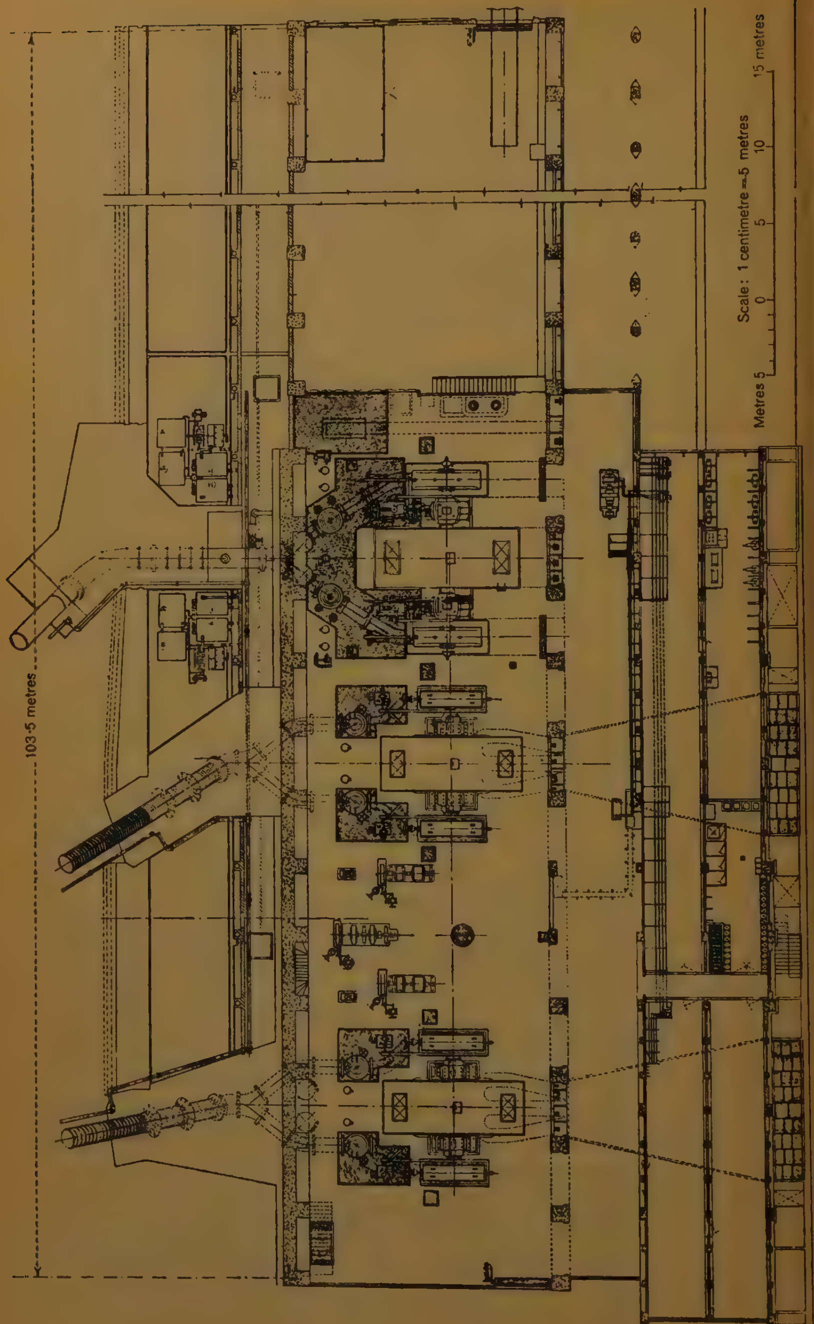
internal protection, the financial importance of keeping such large units in continuous service leads to the use of bituminous enamels, in order to reduce the periods during which the units are out of service. In selecting the enamel, consideration has to be given to the temperature of 163° F. which the steel, painted black with bituminous paint, attains in the sun when the pipe-line is emptied. Although whitewash lowers the temperature by 29° F., it is not sufficiently durable and so aluminium paint in spar varnish is being used to reduce the temperature by 14° F. Provision will also be made to sprinkle automatically the pipe-line if it expands in the sun to an extent which indicates the possibility of incipient " curtaining " of the enamel.

The power-station (*Figs. 9* and *10*, pp. 692 and 693) is located over the former bed of the Pedras river, which now forms the tailrace. It is distant by transmission line 26 miles from the centre of the city or 21 miles from the outer high-tension ring encircling the city. A railroad siding, 2½ miles in length, connects with the main lines of the São Paulo Railway (5 feet 3 inches gauge) at the Cubatão station 9 miles from the Santos Docks. The railroad grade is practically level and pieces exceeding 45 tons in weight can be discharged from the ship by a floating crane on to railway wagons.

The generating units are of the horizontal double-overhung type, having the generator in the middle between the two bearings with the two wheels mounted one on each end of the shaft; the single jet for each wheel is located parallel to and beneath the floor. The normal speed is 360 revolutions per minute, whilst the runaway speed is about 625 revolutions per minute for the complete unit as installed. The rotating parts of the latest unit, the largest yet built of its type, weigh 203 tons and the complete unit, including stop-valves and hydraulic auxiliaries, weighs 650 tons. On each of the units two independent governors are provided, one for each wheel and nozzle, and are adjusted to slightly different timings to avoid the effect of possible resonant action on pipe-line surges. The normal pressure at the nozzles at no load is about 1,020 lbs. per square inch. The theoretical and actual outputs, efficiencies, and heads of a turbine at various discharges are given in *Fig. 11*, p. 694.

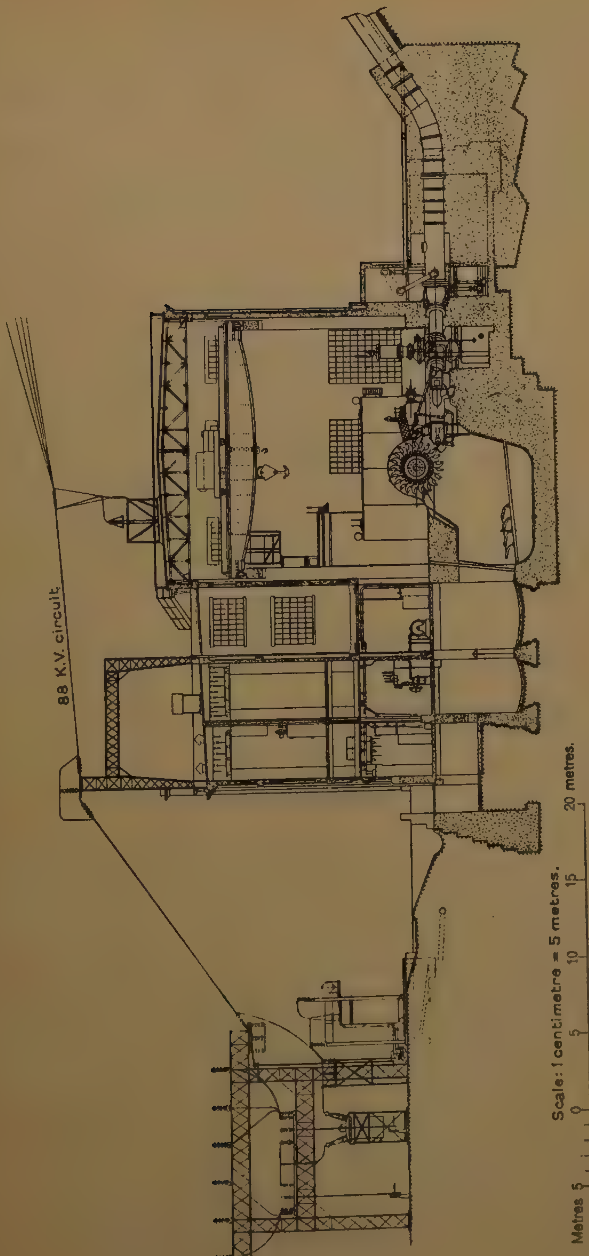
The jets of the earlier and later units are 7½ inches and 9½ inches respectively in diameter at maximum overload, chrome-nickel steel being used for the needle tips and nozzle rings. The velocity of the jet is about 380 feet per second at rated load, the speed of the buckets being, as usual, slightly less than half this. 13 per cent. chrome steel is used for the buckets of the latest unit; the material has to be very resistant and sound and the surface very accurately finished in order to resist the corrosion produced by cavitation under the jet.

Fig. 9.



PLAN OF SERRA DO CUBATÃO POWER HOUSE AND

Fig. 10.



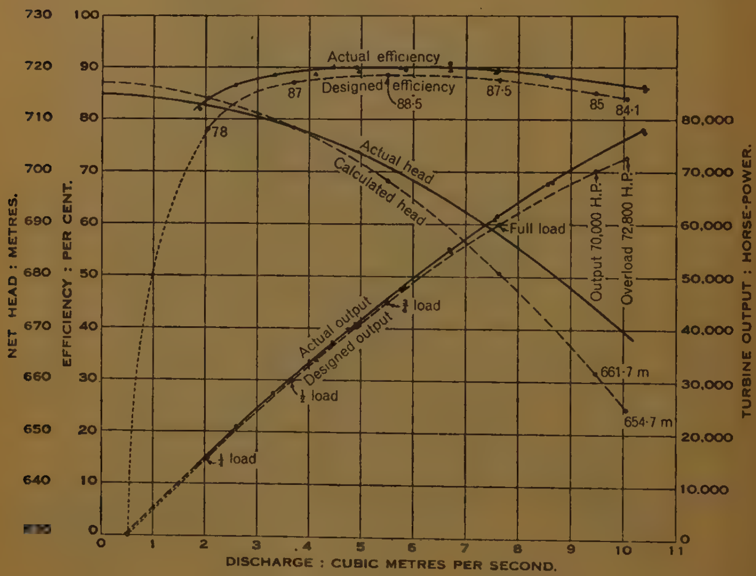
Cross Section of Serra do Cubatão Power-House, Switch-House, and Outdoor 88,000-Volt Station.

The supply of water to each nozzle is controlled by a 29½-inch hydraulically-operated gate valve.

The main bearings of the earlier units are 24 inches in diameter by 72 inches long, and those of the third unit are 27½ inches by 57 inches. The shafts are hollow.

The electrical layout and switching equipment were described in a paper presented in 1929 before the Institution of Electrical Engineers by R. W. H. Couzens.[1] The maximum voltage used in the São Paulo system at present is 88 kilovolts, with a frequency

Fig. 11.



of 60 cycles, whilst that in the Rio system is 132 kilovolts with a frequency of 50 cycles. Copper conductor is used for the 88-kilovolt lines; aluminium cable with steel core is, however, used for a 90-mile 132-kilovolt-line from the Ilha dos Pombos plant to Rio, the maximum and average spans on this line being 4,970 feet and 1,575 feet respectively; these are exceptionally long.

As an example of a modern hydro-electric plant diametrically opposite in design, the Ilha dos Pombos plant on the Parahyba river (*Fig. 1*, p. 679), 90 miles north-east of Rio, may be mentioned. This is a "run-of-river" plant without storage; the river has a minimum flow of about 7,000 and a maximum flow of over 2,000,000 cusecs; the use-

[1] R. W. H. Couzens, "The Serra Hydro-electric Development, São Paulo, Brazil," Journal Inst. E.E., vol. 68 (1930), p. 1291.

ful flow, up to about 22,000 cusecs, is diverted by a low dam into a dredged channel 1½ miles long and returned in a fall of 115 feet to the river at the foot of a rapid. The plant will have a capacity of about 228,000 horse-power, when completed, and is notable for its simplicity and cheapness when compared with the storage plants in parallel with which such run-of-river plants need to operate. It also has three automatic sector-gates of reinforced concrete, the largest of their type, each 147 feet long, 35 feet in radius, weighing 2,200 tons, and raising the water level 24 feet over the sill; these can regulate the pool level to within less than 1 inch.

In all this work it is considered essential, not only to specify and design each part of the equipment for the extreme emergency condition to which it may be subjected, but also to produce these conditions, as far as possible, in the actual acceptance tests. Thus the ruggedness of the plant, as well as its efficiency and satisfactory operation, are proved beyond question.

In pursuing this policy, we test each pipe-line to that point where incipient but scarcely-observable plastic yield is about to occur; this pressure is maintained long enough for a minute inspection of every pipe, joint, saddle pier and anchor block. These tests are at present usually carried out by sections to somewhat different over-pressures.

Similarly, in addition to the more usual tests, each turbine and generator is subjected to the maximum runaway speed which can occur. A similar over-speed test on the generator is always made previously at the factory, in spite of the expense and delay, on each unit of design appreciably different from that of those previously tested. Similarly, each generator is subjected to a dead short-circuit under full-load excitation for 90 seconds.

For testing hydraulic efficiency the Allen "salt velocity" method is adopted as standard, thus avoiding the uncertainties and disputes arising from the use of current meters. The "Gibson" method is also used on low-head installations.

It may be noted that hydro-electric work differs from other construction in that practically the entire design depends on local conditions, with the result that hardly any two plants are alike. A modern steam plant in Buenos Aires, for example, would be very similar to one in London. Hydro-electric installations, on the contrary, must be fitted to and planned on the ground. The variations of climate and river-flow are fundamental and demand long and continuous observations; the selection of the mode of development and of the sites of the corresponding structures cannot be made from a distance. Particularly is this course necessary in a country like Brazil where rainfall, evaporation, run-off and other factors are so variable.

In regions where conditions are more uniform, it is usual to study and estimate the river-flows from the point of view of run-off expressed as a percentage of the rainfall. This method is of little assistance in southern Brazil; in different positions of the basin, only 215 square miles in area, of the first large reservoir of the Serra plant, the percentage of run-off varies from 33 to 83 per cent. averaging 58 per cent. With such variations within a few miles, one must depend on actual gaugings of the river-flows. Government records are scanty or non-existent; therefore a hydro-electric enterprise must maintain its own corps of observers. On the Serra project and in the adjacent region, there are one hundred observers, sixteen recording river-flow gauges, five measuring weirs, thirty-six other gauging stations, seventy-eight rain-gauges, and twenty-two floating evaporation-pans, in addition to other appliances. Many years of patient observation of all these factors are required before sufficiently accurate knowledge of the variations of the available flow is possible.

In high-head projects with large storage reservoirs, evaporation has an importance which is seldom realized. Even though temperatures and winds are moderate and humidity high, the actual evaporation from the water-surface of the first regulating reservoir of the Serra development corresponds to a loss of 38,000 horse-power at 60-per-cent. load-factor; the later steps would bring this loss up to 95,000 horse-power. This loss, however, is more apparent than real as the previous losses by evaporation from the land surface and by transpiration from the vegetation are eliminated by the flooding. The result is probably a net loss in dry years but a net gain in average and wet years. When, however, these large reservoirs are connected by pumping plants which can transfer great volumes of water from one to another, the possible economies of water in operation, by reducing the evaporation, are considerable.

In this region of Brazil, as in other sections of South America, the rainfall and river-flows seem to be more under the influence of the solar cycle than in other parts; records are meagre but indicate that about every 10 or 11 years, just at and following the sunspot minimum, there occur two, or sometimes three, markedly dry seasons with scanty rainy seasons in between. All calculations of available power must take into account these extremely dry periods; otherwise when the successive years of scanty rainfall arrive, disastrous power-shortage will occur.

Time is too limited to discuss in detail these relations of water supply to variations in the solar radiation, as shown by the fluctuations in number, size, location and magnetic polarity of the sunspots and similar indications. It is expected, however, that long, patient

study over many coming years will give eventually results of practical commercial importance; we are ourselves contributing in a small way to one branch of these studies by obtaining accurate records of ocean temperatures by means of thermographs installed on two liners plying between New York and Buenos Aires. These studies have not progressed long enough to determine their utility in practice. It is hoped, however, that by recording through the next group of minimum years, expected about 1944, the observations will assist materially the efforts to predict the following minimum expected about 1955.

It is obviously worth while devoting considerable attention to such studies, if one can be enabled thereby to predict with some confidence when these critical periods are to be expected. During the other more abundant years of the cycle the excess flow may be used for certain services like electric boilers which can be interrupted, whilst the production of steam by fuel can be resumed in the critical periods; by doing this the danger of over-estimating the water supply and of over-selling the production, with its disastrous consequences, is avoided, and spare units can be made to earn much more than their interest charges during development of the load.

One detail of this project which has been studied carefully, is the possible utilization of this chain of lakes and canalized rivers for barge navigation, but for other than economic reasons it will perhaps not be carried out. Although it may appear strange at first sight, thorough investigation has shown it to be commercially practicable to transport freight in bulk from the port of Santos to the city of São Paulo, along the waterfront on these canals; the transfer from coastal plain to plateau, at the edge of the Serra, would be made by ropeway or by barge incline. In two of the dams provision has been made for boat-lifts, and in one case for locks, for transferring barges of a capacity up to 200 tons. In any case provision will be made on the plateau for transferring smaller barges of a capacity up to 30 tons each.

In all work in warm countries, the engineer must watch and guard against insanitary conditions, as the health of the entire working force depends on his appreciation of the problems of sanitation. In this particular work in Brazil, exceptional success has been obtained by our Brazilian medical force in combating malaria and intestinal parasites, which are the most common diseases.

Malaria exists everywhere in the coastal plain (except in the large cities) and in a large part of the interior, being absent, however, in the region along the crest of the Serra and the adjacent plateau. Unless care is taken, and malarial subjects are eliminated by careful

examination of all incoming workmen, epidemic malaria, usually of the benign tertian type mixed with the malignant tertian, will paralyse the progress of the work and raise costs proportionately. Even in that portion of the region which is free from malaria, the bringing in of large numbers of labourers, unless those infected with latent malaria are carefully weeded out, will cause malaria to appear. If this happens in the region near an artificial reservoir, it is usually attributed unjustly to the existence of the reservoir.

In the Serra work success has been obtained by simple practical means; no quinine or other prophylactic is given nor are windows in the barracks screened, but reliance is placed mainly on careful inspection not only of the usual draining and oiling of the camps and adjacent areas but also of the working force, especially of all new comers. The most important precaution is to have the camp watchmen require every man who wishes to absent himself from work and to stay in his quarters, to go first to the doctor; if the latter finds the least indication of temperature above normal, he makes a blood-smear test and in a few minutes the microscope shows positively whether the man has incipient malaria. If he has, he is sent to the hospital, treated for 30 days and kept there until the microscope shows no more parasites in the blood.

There are many important problems regarding malaria which would interest you, such as the impossibility in the forest of eliminating mosquitoes which may transmit malaria, because they thrive in the parasitic plants on the trees; the effect of temperature on the health and development of a thriving city like São Paulo (which is free from malaria), and the gradual invasion of malaria into previously healthful zones. I am afraid that I have not time to discuss these or other diseases such as hookworm, which affects nearly every rural inhabitant in warm, moist regions, as found in Brazil.

The time available has only allowed me to give you an outline of so extensive and varied a project: I hope, however, that I have conveyed to you some idea of its principal features, the natural conditions, simple and yet so favourable, of this portion of Brazil, the many interesting but only partially-solved problems of long-range weather-foreshadowing, of pipe-line design and of sanitation, and especially the almost unlimited opportunities for hydro-electric development of which this Serra do Cubatão plant is typical.

The Meeting concluded with a vote of thanks proposed by Sir Richard Redmayne, Past-President, seconded by Mr. F. E. Wentworth-Sheilds and supported by H.E. Señhor Dr. Régis de Oliveira, the Brazilian Ambassador.

VERNON-HARCOURT LECTURE, 1935–36.[1]

"Tidal and River Models."

By Professor ARNOLD HARTLEY GIBSON, D.Sc., LL.D., M. Inst. C.E.

IN many engineering problems the forces and actions involved are so complex as to defy any attempt at an analytical solution, and the only method of attack is by experiment. This is especially the case where the interaction between a body and a fluid in relative motion is involved, as, for example, when an aeroplane or ship is in motion, or when water is flowing over a weir or through sluices, or over the erodible bed of a river, in which further complications may be caused by the introduction of obstructions or of training works.

Because of the cost and difficulty of experiment on the full scale in such cases, considerable attention has been paid to the possibilities of using small-scale models, and of extending the results of tests on these to the full-scale prototype. The scale model is now accepted by the aeroplane designer and naval architect as an indispensable instrument of investigation. The technique of ship-model and wind-tunnel investigations has been highly developed, and although the results can seldom be extended to the full scale without some correction, it is possible in most cases to estimate the magnitude of this correction.

During recent years much work has been done on models reproducing the flow over weirs and through sluice-gates, etc. Here again, comparison with full-scale results shows that if suitable precautions are taken the model gives a reliable indication of the behaviour of the original.

River models appear first to have been used in France, where, in 1875, in connection with an inquiry into the measures to be taken to improve the river Garonne between Bordeaux and the sea, a model was constructed to investigate the validity of certain principles of river training formulated by M. Fargue. This model, which was to a longitudinal scale of 1 : 100, was about 60 metres long and 1 metre wide. Although everything in the experiment was arbitrary,

[1] This Lecture was delivered at a meeting of the Association of London Students, and repeated before Local Associations at Belfast, Birmingham, Bristol, Glasgow, Leeds, Manchester, and Newcastle.

including the relationship of horizontal and vertical scales and the duration and volume of flow, it did give results of considerable practical value.

The first real attempt to apply scientific principles in the design of such models was made in 1885, when Osborne Reynolds, then Professor of Engineering in the University of Manchester, constructed two small-scale models of the upper estuary of the Mersey, having horizontal scales of 1 : 31,800 and 1 : 10,600, and vertical scales of 1 : 960 and 1 : 396, in connection with an examination of the possible effect of the proposed Manchester Ship Canal on the regime of the estuary. In these models, Reynolds first introduced the time element and the general idea of dynamical similarity.

These experiments showed that the characteristics of the real estuary reproduced themselves in the models after a corresponding number of tides, and the work resulted, in 1888, in an investigation of the general question of the use of such models, in which Reynolds co-operated with a committee appointed for the purpose by the British Association. The work was devoted mainly to an examination of the behaviour of models of the same hypothetical estuary, of symmetrical shape but to different scales. As a result of the investigation the committee reported in 1891 to the effect that "it would seem, therefore, that by carefully observing certain (stated) precautions the method of model investigation may now be applied with confidence to practical problems."

In the meantime, in 1886 Mr. L. F. Vernon-Harcourt, M. Inst. C.E., the founder of this series of Lectures, constructed a small tidal model to examine the effects of a series of alternative schemes which had been proposed for the improvement of the estuary of the Seine between Rouen and the sea.[1] This model had a horizontal scale of 1 : 40,000 and a vertical scale of 1 : 400. Starting with the bed of the model in a state approximating to that of the river in 1834, the results of improvements made in the estuary since that time are said to have been closely reproduced. The experiments indicated that none of the proposed schemes would accomplish the desired results, a conclusion which was confirmed by an independent model investigation carried out between 1890 and 1895 at Rouen under the auspices of the French Government.

During the passage of the Manchester Ship Canal Bill through the Parliamentary Committees in 1883 and 1884, it was at first proposed that the canal should enter the estuary near Runcorn and that the

[1] L. F. Vernon-Harcourt, "The Principles of training Rivers through Tidal Estuaries, as illustrated by Investigations into the Methods of improving the Navigation Channels of the Estuary of the Seine." Proc. Roy. Soc., Vol. 45 (1889), p. 504.

upper estuary below this point should be regulated by training walls. The opponents of the scheme, who suggested that such works would cause siltation of the lower estuary and would adversely affect the navigable channel between Liverpool and the Mersey bar, were successful in their opposition, and the canal was therefore continued along the southern bank to deep water at Eastham. In 1889, Mr. Vernon-Harcourt [1] constructed a small model of the estuary between Runcorn and the Mersey bar to a horizontal scale of 1 : 30,000 and a vertical scale of 1 : 500, in order to examine what would have been the effect of the original scheme. He found that this was as had been predicted. Considerable shoaling was produced in the model in the vicinity of Liverpool itself, and the outer channel was seriously affected.

Although the experiments of Reynolds and Vernon-Harcourt indicated definitely the value of the scale model as a means of attacking the problems involved in the regime of tidal and river channels, they did not by any means give a complete solution. The results were qualitative rather than quantitative. The effect of the size and density of the bed materials was only examined in a very tentative manner, and no definite comparisons were drawn between the channel depths in the model and in nature.

No systematic attempt to extend this work was made for some years, but in 1898 a permanent River Hydraulic Laboratory, the first of its kind specifically designed for the investigation of river hydraulics, was founded at Dresden under the control of Professor Engels. Following this, similar laboratories were established in rapid succession on the continent of Europe. The work done in them covers a very wide field. Models of harbours in tidal waters have been extensively investigated with a view to determining the best form and position of entrance, the best means of preventing siltation, and the magnitude and direction of ebb and flood currents. Problems involved in the filling of large locks, and in the design of intake-structures for hydro-electric stations and of discharge-structures on navigable waterways, have received much attention, and many investigations into the deepening and training of river-channels have been carried out. Much information regarding these institutions—now more than twenty in number—and the work on which they have been engaged, is to be found in the publication "Hydraulic Laboratory Practice" issued in 1929 by the American Society of Mechanical Engineers. More recently, institutions of the same type have been established in the U.S.A., where there is now a

[1] L. F. Vernon-Harcourt, "Improvement of the Maritime Portion of Rivers, including their Outlets." 5th International Congress on Inland Navigation, Paris, 1892. Question 10.

Waterways Experiment Station at Vicksburg, Mississippi, operated by the Government, in which a considerable amount of work has been done during the past 4 or 5 years, especially on problems connected with the training of the Mississippi river.

Although Great Britain may perhaps claim to be the birthplace of the scientific use of such models, it has no permanent laboratory specifically laid out for this type of investigation. In spite of this, however, a good deal of work has been carried out of recent years, especially in connection with tidal estuary problems. An investigation was initiated in 1926 by the Severn Barrage Sub-Committee of the Committee of Civil Research, who decided to examine the probable effect of a proposed tidal-power barrage across the Severn estuary by means of a scale model constructed and operated at the University of Manchester. This proved to be the forerunner of a number of model investigations of the type in this country. Models of the Mersey, Dee and Humber have since been investigated; the Great Ouse Catchment Board have recently constructed a model of the Great Ouse and its outlet to the Wash; preparations are at present in hand for a model covering some 20 miles of the river Mersey below Northenden; and work has just been completed on a model of the Rangoon estuary and river.[1] This model, in which the effects of coastal erosion and of the monsoon gales have been incorporated, represents probably the most interesting investigation of this type yet attempted.

Dynamical Similarity.

In any model involving motion, certain fundamental relationships must be satisfied if it is to give a true reproduction of the behaviour of the original. In the first place, the model should be geometrically similar to the original. It is also necessary that all speeds in the model should be so related to those in the original that all corresponding pairs of forces in model and original, whether due to gravity or called into play by the motion, shall be in the same ratio. When this is the case the model is said to be dynamically similar to the original and the speeds of model and original are called "corresponding speeds."

This may be simply illustrated by reference to a rotating fly-ball governor. The weights are acted upon by the force of gravity and by centrifugal force, and the inclination of the arms depends on the ratio of these forces and therefore on the peripheral speed V, the tangent of the vertical angle being V^2/gR, where R denotes the radius of the ball path. Now a rotating small-scale duplicate of this is

[1] By Sir Alexander Gibb and Partners, at University College, London.

only a true model if its speed is such that its arms are at the same inclination as in the original. For this, v^2/gr (the small letters referring to the model), must be the same as V^2/gR in the original. It is therefore necessary that $\frac{v}{V} = \sqrt{\frac{r}{R}}$, so that in this case corresponding speeds are in the ratio of the square roots of corresponding dimensions.

When fluid motions are involved, forces are produced whose magnitudes depend upon the viscosity μ and the density ρ of the fluid, and the corresponding speeds should be such that all corresponding pairs of forces due to these factors should have the same ratio. If gravity also affects the phenomena, this ratio should be the same as that of corresponding pairs of forces due to gravity. At these speeds all lines of flow and any wave-formations, or the shapes of any free surfaces, will be similar.

The ratio of the corresponding speeds depends upon the type of model. Wherever flow takes place under the action of gravity, as, for example, in the discharge over a weir, or where surface waves, whose form depends essentially on the force of gravity, are formed, corresponding speeds must be in the ratio of the square roots of corresponding dimensions.

There are, however, cases in which forces due to fluid friction are all-important, and in which gravity has no effect on the phenomenon. For example, the lines of flow around the hull and also the resistance of an airship or a deeply-submerged submarine are independent of the force of gravity. So is the resistance to flow through a pipe and the distribution of velocities over its cross section. In such cases the density ρ and viscosity μ of the fluid are the only potent factors, and theory shows that corresponding speeds must be such as to make the product $vl\rho/\mu$ the same for original and model. Here l denotes some definite linear dimension, such as the diameter of the pipe or the length of the submarine. The expression $vl\rho/\mu$ or vl/ν * is known as the "Reynolds number." If the same fluid is used in the model as in the original, the first type of model (in which gravity-effects are important) requires

$$\frac{v}{V} = \sqrt{\frac{l}{L}},$$

while the second type (in which viscosity-effects are important) requires

$$\frac{v}{V} = \frac{L}{l}.$$

* ν is called the "kinematic viscosity" of the fluid.

It is evidently impossible to satisfy both these requirements simultaneously, so that where, as is often the case in practice, both gravitational and viscous forces are involved, it becomes impossible to choose corresponding speeds which will give exact similarity as regards all the forces called into play.

Fortunately, in most cases one or other factor is much the more important, and then the corresponding speeds to be adopted are those which produce similarity of the forces due to that factor. The forces due to the other factor will now not be similar in the model and the original, and this will introduce a scale-effect, which, if large, will invalidate an extension of the model results to the full scale. If not large, or if the magnitude of the effect is known, the results can be extended to the full scale. Thus in ship-model tests it is highly important to have similar wave-formations accompanying ship and model, and this requires that the corresponding speeds shall be proportional to the square roots of their respective lengths. Since there are viscous forces in operation causing skin-friction and affecting the lines of flow in the immediate vicinity of the hull, this choice of speeds will lead to dissimilarity in these respects, and will thus cause a scale-effect, for which, however, a correction can be made.

In any kind of model, if the scale-effect is not to be serious the type of motion must be the same as in the original. As is well known, two distinct types of fluid motion are possible—laminar or streamline, and turbulent.

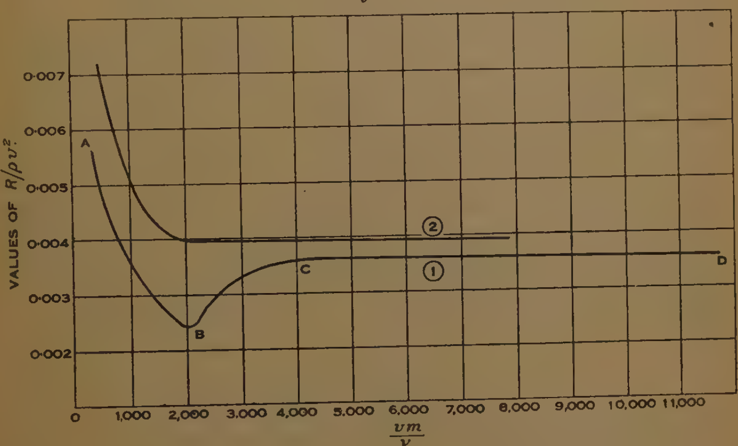
The first of these tends to occur at low speeds, with fluids of high viscosity and in flow through channels of small dimensions—that is, at low values of the Reynolds number. In it the fluid forces are entirely governed by the viscosity of the fluid. Turbulent motion sets in at some definite value of the Reynolds number and persists at all higher values. In such flow inertia forces predominate, and viscous forces become less and less important as the turbulence increases until, when the resistances become proportional to v^2, viscosity ceases to have any further modifying effect on the lines of flow, or on the resistance.

In most of the hydraulic problems in which models are used, and certainly in all river and tidal problems, the flow in the original is turbulent, and it is therefore essential that the model shall be of such a size that its Reynolds number under operating conditions is in the range of turbulent flow. If it is, and if viscous or skin-frictional forces are of secondary importance, any scale-effect due to this factor will be small, even though the Reynolds number of the model is much less than that of the original.

In this connection the curves of *Fig. 1* are of interest. They

represent the results of one of a series of experiments to determine the resistance and the critical velocity of flow in small rectangular channels of uniform cross section recently carried out in my laboratory by Mr. Jack Allen, Assoc. M. Inst. C.E.[1] The curves show values of $R/\rho v^2$ as ordinates, where R denotes the resistance per unit area of wetted surface and ρ the density of the fluid, plotted on a base of Reynolds numbers, vm/ν.* Curve (1) represents the results when water was led into the channel through a converging inlet giving definite streamline motion at the entrance, and curve (2) shows corresponding results obtained without the stabilizing influence of the convergence.

Fig. 1.



In curve (1) AB represents the regime of streamline or laminar flow, BC the transitory range during which flow is very unstable, while CD shows turbulent flow with the resistance proportional to v^2. Point B marks the higher critical velocity at which stabilized laminar flow breaks down, and which occurs at a Reynolds number of 2,100. In curve (2), where the stabilizing effect of the convergent entry is absent, the change to turbulent flow takes place earlier, at a Reynolds number of about 1,500. The latter state of affairs is more likely to apply in any river model. The results show that even in a model of a straight uniform reach of a river or canal turbulence will be attained if the Reynolds number exceeds 1,500. Thus, when the hydraulic

[1] J. Allen, "Streamline and turbulent flow in open channels." Phil. Mag., Vol. 17 (1934), p. 1081.

* Here m denotes the hydraulic mean depth of the section, that is, the area of the cross section divided by the length of the wetted perimeter.

mean depth is 1 inch (2·54 centimetres), since at normal temperatures ν has a value of approximately 0·010 in c.g.s. units, any velocity greater than 5·9 centimetres per second (0·2 foot per second) will give turbulent motion. In a model of any actual river with irregular bed and sides the necessary velocity would be appreciably less than this.

Osborne Reynolds, as a result of experiments on models of hypothetical estuaries of simple triangular form to different scales, concluded that for the motion in the model to be turbulent and to be able to produce similar sand formations in each case, the value of h^3e should not be less than $0{\cdot}09\sqrt{\frac{30}{H}}$. Here h denotes the maximum tidal range in the model in feet, H the corresponding range in the estuary and e the ratio of the vertical and horizontal scales. Thus, in a model of an estuary with a tidal range of 30 feet, the model having a horizontal scale of 12 inches to 1 mile (1 : 5,280), calculation shows that a vertical scale of 1 : 200 will just satisfy this criterion. Here $h = 0{\cdot}15$; $e = 26{\cdot}4$. A vertical scale of 1 : 250 would give a criterion of 0·0366, and one of 1 : 150 would give a criterion of 0·28. Actually in a normal estuary the irregularities of the conformation both of sides and bottom cause turbulent flow to be set up at lower velocities than in those for which the criterion was deduced by Reynolds, so that somewhat lower values can safely be used in practice.

Distortion of Scale.

In some types of hydraulic model it is essential that the horizontal and vertical scales shall be the same. This is the case in models of weirs or sluices or in any model in which a standing wave is produced, since the form of the free surface depends entirely upon the velocity of flow, which in its turn depends only on the head and therefore on the vertical scale, so that if horizontal distances on the profile are to correspond in model and original the horizontal and vertical scales must be identical.

In the case of a river or estuary model it is almost impossible in practice to adopt the same scale for both horizontal and vertical distances. More especially in tidal models, where, from the nature of the case, considerable areas have to be incorporated, the horizontal scale reduction must be considerable if the model is to be kept within reasonable dimensions, and a scale of more than 18 inches to 1 mile (1 : 3,520) is unusual, a more common ratio being about 1 : 8,000.[1]

[1] In the Severn model the horizontal scale was 1 : 8,500 and the vertical scale finally adopted was 1 : 200.

If this latter ratio were also adopted for the vertical depths in a model of an estuary having a tidal range of say 33 feet, the range in the model would only be 1/20 inch and the current-velocities, which are proportional to the square root of the depth, would only be about 1/90 of those in the estuary. Under these circumstances the motion of the water would certainly not be turbulent as in the estuary. To avoid this difficulty the vertical scale is made much larger than the horizontal scale, and in practice the vertical exaggeration of scale usually ranges from about 5 : 1 to 40 : 1. By making the vertical scale 1 : 200, the tidal range in the case mentioned would be 2 inches and the current velocities would be 1/14 of those in the estuary.

This distortion of scale means that all gradients in the model are greater than those in the river or estuary, and at first sight it might appear that this would seriously reduce the usefulness of such models. Actually, however, some such exaggeration of vertical scale appears to be natural, and rather an advantage than otherwise except in special cases. It is a matter of observation that there is an exaggeration in the vertical scale of small natural streams, as not only are their longitudinal gradients greater than those of large rivers having the same bed material, but they also have steeper banks and greater depths compared with their breadths.

Recent investigations [1] suggest that such an exaggeration of scale is indeed necessary if similar regimes are to be established, and indicate that it has some definite value which will give similarity of regime if the bed material is the same in the model as in the original. The relationship proposed is, in effect,

$$\frac{R}{r} = \left(\frac{L}{l}\right)^{\frac{2}{3}},$$

where R denotes the hydraulic mean depth and L the length.

With this exaggeration of scale, there may be points in a model at which, in order correctly to reproduce the slopes in the original, the slopes would need to exceed the natural angle of repose of the bed material, and at such points accurate reproduction would be impossible. The actual slopes of the sandbanks in most estuaries and rivers are, however, very slight, and experience shows that the areas over which the angle of repose would be exceeded in the average model do not amount to more than a small fraction of the whole. In such cases, if thought desirable, a slight stiffening of the bed material at those points with an admixture of clay will allow the required slope to be maintained.

[1] Gerald Lacey, "Stable Channels in Alluvium." Minutes of Proceedings Inst. C.E., vol. 229 (1929–30, Part I), p. 259.
"Uniform Flow in Alluvial Rivers and Canals." *Ibid.*, vol. 237 (1933–34, Part I), p. 421.

In a model intended to investigate the effect of such training works as spur dykes or groynes, which cause an intense scour and comparatively steep slopes in their vicinity in the river itself, the distortion of scale should be as small as possible. In general the area of local scour will be exaggerated in the model, owing to the fact that after the material is scoured out to its natural angle of repose any further scour causes more surface material to slip down the scoured incline.

Experiments on the effect of spur dykes on a model of the Rhine [1] having a natural scale of 1 : 200 have shown an excellent agreement with the results observed in the river. Experiments with a vertical exaggeration of 2·5 showed the same maximum depths of scour, but over a somewhat greater area. Experiments have been made on the effect of spur dykes in a model of the Mississippi,[2] with a horizontal scale of 1 : 1,000 and a vertical scale of 1 : 125, and the results obtained have since been confirmed in the river. The general agreement is said to be very close.

Time Scale Ratio.

In a tidal model the correct reproduction of the tidal wave is an essential factor. Since the velocity of such a wave is proportional to the square root of the depth of water in which it is propagated, corresponding speeds in model and estuary must be in the ratio $\sqrt{H/h}$ of the square roots of corresponding depths. If then the wave is to travel corresponding distances l and L of the model and estuary in corresponding times, these times must be in the ratio

$$\frac{t}{T}=\frac{l}{L}\sqrt{\frac{H}{h}}.$$

This ratio gives the necessary time t of a tidal period in the model corresponding to the time T, which is about 12 hours 25 minutes, in nature. Thus in a model having a horizontal scale ratio of 1 : 5,280 and a vertical scale ratio of 1 : 200, the time ratio would be 1 : 373 and the tidal period 119·8 seconds, so that the equivalent of 1 year's tides would be reproduced in 24 hours.

Corresponding volumes are in the ratio HL^2/hl^2, and corresponding volumes per unit time in the ratio $LH^{\frac{3}{2}}/lh^{\frac{3}{2}}$, so that the necessary discharge of any river entering the model can be computed if the discharge of the original is known.

Bed-Materials.

When the question arises of the correct bed-material for a model

[1] J. R. Freeman, "Hydraulic Laboratory Practice," p. 219. New York, 1929.
[2] Paper 15, U.S. Waterways Experiment Station, Vicksburg, Jan. 1934.

having an erodible channel, theory ceases to be of much assistance and we have to rely on experiment. At first sight it would appear to be necessary for the size of the bed-material to be scaled down in proportion to the vertical scale-reduction of the model, and, indeed, if the model is to represent to scale the surface-roughness of the bed, the grain-size should be reduced in this ratio. Such a material, however, would be an almost impalpably fine powder which, as Stokes's Law tells us, would not obey the same laws of fluid resistance when in motion as the larger grains in the estuary, and would certainly not reproduce the erosion and deposition of its bed.

Fortunately, experiment shows that this question of similarity of surface-roughness is of very small importance in a model of an actual river or estuary, since the resistance to flow due to eddies and cross currents caused by curves and major irregularities in the bed and sides is usually overwhelmingly greater than the skin-friction due to the texture of the bed. In an estuary, the tidal wave, after attaining its maximum range, is rapidly damped out by these various resistances as it travels upstream. My colleagues and myself have been responsible for a number of tidal models, in each of which the size of the bed-material has been of the same order as that in the estuary, and if the effect of this roughness had been important the tidal range in the upper estuary would have been reduced relatively much more rapidly in the models than in nature. Actually in every case the modification in the range of the tides, the levels of high and low water, the periods of ebb and flow and the duration and velocity of the tidal currents have been reproduced at every point in the model, with a high degree of accuracy.

As regards the movement of the bed-materials by the scour of the currents, the best material is that which reproduces most accurately that of the bed of the river or estuary, and this can only be determined by experiment.

The early investigators tried various materials. Thus Vernon-Harcourt in his model of the Seine used in turn silver sand, powdered sulphur, coal-dust, pumice, and bath brick, finally relying on a very fine sand having a small admixture of peat, from the Bagshot beds. Osborne Reynolds, after experimenting with various materials, finally used fine Calais sand in his experiments on the Mersey and other models. Neither Vernon-Harcourt nor Reynolds gave any data as to the sizes of their sands or their relationships to the sizes of sand in the respective estuaries, but I have been able to obtain a sample of the sands used in each case. In the Mersey models the sand has a mean diameter of 0·0065 inch, as compared with a mean diameter of 0·0082 inch for a number of samples from the estuary itself, the ratio of these diameters being 0·8. In the Seine model the

diameter was 0·0067 inch, or sensibly the same as that of the sand used by Reynolds.

These sands were chosen, not as the outcome of any specific investigation into the effect of grain-size, but apparently because they happen to have been the finest sands easily obtainable. Although they appear to have given good results, it is possible that a sand either somewhat finer or coarser might have proved even more suitable. It is interesting to note, however, that Reynolds used the same size of sand in all his models, and although the scales were varied over a fairly wide range—the longitudinal scale between the limits 1 : 39,000 and 1 : 12,000, and the vertical scale between 1 : 170 and 1 : 960—he does not remark on any marked discrepancy between the results. Indeed, he remarks on the general similarity between the results obtained by the different models. So far as his results go, they tend to indicate that a variation of sand size with the scale of the model is not, within limits, a very important factor.

This view receives some indirect confirmation from the results of experiments at Karlsruhe [1] on the scour at bridge piers, in which an increase in the size of sand grain from 0·02 inch to 0·06 inch did not result in any appreciable difference in the scour. Further confirmation is afforded by the experiments of Messrs. A. D. D. Butcher and J. D. Atkinson [2] on the scour at the end of the floor of models of the Esna sluice-dam, where identical slopes were formed with models of scales 1 : 100 and 7 : 100, using the same sand in both.

In order to obtain further information on this point, in the course of the preliminary work on the Severn model at the University of Manchester, thirteen different bed-materials were tested. These consisted of powdered pumice, silica sands, and emeries, of different grain-sizes, some finer and others coarser than the sand in the estuary.

In each case the bed of the model was moulded to represent the state of affairs shown on an Admiralty chart of 1849. The model was then operated for the equivalent of 78 years, after which it was surveyed and compared with the charts of 1927. The detailed results of these experiments have been published elsewhere.[3] Broadly speaking, they showed that so long as the material was not too large to be moved by the currents, the general conformation of

[1] J. R. Freeman, "Hydraulic Laboratory Practice," p. 201. New York, 1929.

[2] "The Causes and Prevention of Bed Erosion, with special reference to the Protection of Structures Controlling Rivers and Canals." Minutes of Proceedings Inst. C.E., vol. 235 (1932–33, Part I), p. 175.

[3] A. H. Gibson, "Construction and Operation of a Tidal Model of the Severn Estuary." H.M. Stationery Office, 1933.

the channels and sandbanks was substantially the same with all the materials. They indicated a fairly definite relationship between grain-size and density and the heights of the resultant banks and depths of the channels in the model, these being inversely proportional to $d^{0\cdot287} \times \rho^{0\cdot262}$ and directly proportional to $r^{0\cdot15}$, where d denotes the mean diameter of the grains, ρ the effective density in water, and r is a measure of the angularity of the grains, being the ratio of the longest to the shortest diameter. The best overall agreement was obtained, not with the finest material tested, but with a sand having a mean diameter of 0·007 inch, which is about 25 per cent. less than the average mean diameter of the sand in the estuary itself.

The foregoing experimental result, namely, that in these models sands of approximately the same size as in the estuaries are in fact eroded and scoured to the same general conformation as in their prototypes, is in line with the results of investigations on the movement of bed materials by Mr. E. C. Thrupp and others, and on scouring and siltation in Indian and other rivers and canals by numerous observers. Mr. Thrupp found that, in streams of different depths, movement of a sandy bottom took place when the mean velocity was proportional to the square root of the depth.[1] Mr. R. G. Kennedy and others [2] have found that rivers and canals of similar cross sections but of different sizes, having the same bed-materials, are subject to similar scour or siltation if the mean velocity is proportional to h^m, where h denotes the depth and where the value of m is in the neighbourhood of 0·5, varying from about 0·45 to 0·65 with the fineness of the bed-material. A more recent investigation by Mr. Lacey indicates that the velocity should be proportional to the square root of the hydraulic mean radius R, which, if the width is large compared with the depth, is sensibly the same as the depth. All this goes to show that close similarity of scour and deposit is to be expected with the same bed-material if the mean velocity is proportional to the square root of the depth. As this is exactly the ratio of velocities adopted for purely hydrodynamic reasons in tidal models, the evidence would indicate that material of approximately the same grain-size and density as that comprising the moving sandbanks in the estuary should logically be used for the model.

Now, the conventional theory of the forces exerted by a flowing stream on an exposed body indicates that for similarity the size of

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[1] "Flowing-Water Problems." Minutes of Proceedings Inst. C.E., vol. clxxi (1907–8, Part I), p. 346.

[2] *Ibid.*, vol. cxix (1894–5, Part I), p. 281; vol. 223 (1926–7, Part I), p. 279.

the body in the model should be reduced according to scale, and recent experiments by Messrs. Butcher and Atkinson [1] on the forces acting on models of cubical blocks of 1 metre side to scales of 1 : 100 and 7 : 100, show that this is the case. The apparent discrepancy between this conclusion and the experimental evidence which has been educed in the case of the movement of bed-materials of small size needs explanation. It is suggested that this is to be found in the fact that whereas in the impact on a large body the forces involved depend only on the mean velocity of the stream, and vary as the square of that velocity, the forces acting to produce motion of the particles of a sand bed depend on the activity of the erosive layer immediately overlying the bed—the so-called boundary-layer—which is affected by factors other than the mean velocity of the stream.

Although our knowledge of the exact state of affairs in this layer is very incomplete, we do know that, even with turbulent flow in the main stream, there is a very thin film adhering to the solid boundary in which the motion is laminar, and that outside this the motion gradually changes from laminar to become fully turbulent at the outside of the boundary layer. At the surface of contact of the water with the boundary the velocity is zero, and it increases rapidly to attain a value at the edge of the boundary layer of from 60 to 90 per cent. of the mean velocity of flow in the stream, the ratio depending on the depth of the stream and the roughness of the bed. This value is what is commonly but inaccurately called the bottom velocity.

In the outer part of the boundary layer the activity is intense. Eddies are being formed and shed off, and in consequence there are rapid local fluctuations of pressure at the outside of the laminar layer. These fluctuations depend not only on the mean velocity but also on the rate of change of velocity in the vicinity. Now, since corresponding velocities in estuary and model are in the ratio $\sqrt{H/h}$ while corresponding depths are in the ratio H/h, the rate of change of velocity with height is greater in the model than in the estuary in the ratio $\sqrt{H/h}$. If, then, the velocity increases from zero at the surface of contact there will be some definite small distance from the bed at which the velocities will be the same in estuary and model. At smaller heights they will be greater in the model, and at all greater heights they will be greater in the estuary. It would follow from this that there is some small but definite height near the bed over which the pressure-fluctuations may be expected to be of the same order in model and estuary.

[1] *Loc. cit.*

It would appear that the movement of sand grains is mainly due to these fluctuations. Sand grains are moved not only by being dragged along the bottom but also by being lifted and then dropped, progressing in a series of hops. The lifting force is provided by the local negative pressure accompanying the passing of an eddy over the grains. If, then, there is the same order of pressure fluctuations near the bed in estuary and model, the same size of sand grain will be equally liable to motion in both.

This would also explain why, within limits, erosion varies very little with grain-size if the grains are small, since an increase in grain-size by increasing the roughness of the surface increases the activity in the boundary layer and thereby increases the forces available to produce motion.

Silt Problem.

Any model of an estuary in which trouble is caused by the deposition of silt must include this effect. Since the silt is usually brought down by the rivers feeding the estuary, an analysis of the percentage of silt carried in the river-water under various conditions of flow is required, and this percentage is to be added to the river-water supply in the model. Silt samples are also required at various points in the estuary, and on starting up the model sufficient silt is to be added to the estuary water to give the approximate concentration at these points. The colloidal silt brought down by the rivers tends to coagulate and to settle, forming silt banks where the fresh river-water meets the saline sea-water, and this effect should also be reproduced if the material is to be deposited at the same place in the model as in the estuary.

Now corresponding times in model and estuary are in the ratio $l\sqrt{H}/L\sqrt{h}$, whilst corresponding depths are in the ratio h/H, and since corresponding particles of silt in model and estuary should sink through corresponding depths in corresponding times, it follows that the actual rate of fall of a particle in the model should be greater than that in the estuary in the ratio $hL\sqrt{h}/Hl\sqrt{H}$ $(= Lh^{\frac{3}{2}}/lH^{\frac{3}{2}})$. Thus with a value of $L/l = 5{,}280$ and $H/h = 200$, the rate of fall should be 1·9 times as great in the model.

This greater rate of fall can be produced by increasing the size of the particles, either by using silt of greater coarseness or by using some coagulating medium more effective than the salts in sea-water. In the models constructed by us at Manchester, the colloidal silt from the estuary itself has always been employed and alum solution has been used as the coagulating medium. Experiments are made with samples of sea-water from the seaward end of the estuary to

determine the rate of deposition of the silt, and further experiments are carried out with alum solution to determine the necessary concentration of this to produce the required rate in the model. On starting up the model the water at the seaward end is brought up to the required concentration. Since river-water is constantly being added at the upper end of the estuary and discharged over a spillway at the seaward end so as to maintain a constant mean level, the concentration would gradually get less unless fresh alum solution were added; sufficient of this is therefore fed in at a constant rate at the seaward end to give the required concentration to the amount of water being discharged over the spillway.

River Models.

In a tidal model, since flow takes place alternately up and down the model on the flood and ebb tides, the longitudinal gradient is settled definitely by the ratio of the vertical and horizontal scales and is greater than that in the estuary in this ratio. In a river model, however, with flow in one direction only, it is possible to make the gradient independent of the distortion of scale, and a technique has been developed in some continental laboratories in which the bed-material used is considerably coarser and somewhat less dense than that in the river. Powdered lignite has been found to give good results. At the same time the slope of the bed is increased so that its gradient is relatively greater than that corresponding to the distortion of scale. This is found to prevent the formation of sand ripples which are formed with some combinations of grain-size and velocity of flow. The correct gradient for any size or density of bed-material is determined by experiment.

Much work has been done on river models during the last 5 years or so in the U.S. Waterways Experiment Station at Vicksburg, Mississippi. Here a technique has been evolved based on the validity of the relationship known as DuBoys's law,

$$T = wds,$$

where T denotes the tractive force in lbs. per square foot exerted by flowing water on its bed, w the weight of water per cubic foot, d the depth of the water, and s the slope of the water-surface. Tests on the actual material of the river-bed are made in a flume whose slope can be varied. In the tests the slope is gradually increased, keeping the water-surface parallel to the bed of the channel until general movement of the bed-material occurs. The tractive force then calculated from DuBoys's formula is termed the "critical tractive force" for the material in question. Similar experiments are

carried out for the material to be used for the model, which may be of any convenient grain-size.

Water-surface profiles from the river are then examined, and by inserting the observed slopes and the critical tractive force in the formula, the stage at which general bed-movement begins in the river is determined. The product of this depth and the vertical scale gives the stage at which similar bed-movement should begin in the model. Using this depth and the value of T for the model bed-material gives the required slope for the water-surface in the model.

Having chosen the horizontal and vertical scales, the value of the hydraulic mean radius R is computed and the appropriate velocity of flow for the model is deduced from Manning's formula

$$v = 1{\cdot}486\ R^{\frac{2}{3}}S^{\frac{1}{2}}/N,$$

where S denotes the slope and where the constant N is, as a result of experience, taken as being the same for river and model.

Then $$\frac{v_m}{v} = \left(\frac{R_m}{R}\right)^{\frac{2}{3}}\left(\frac{S_m}{S}\right)^{\frac{1}{2}},$$

giving the ratio of corresponding velocities and hence enabling the correct discharge for the model to be calculated.

This method is frankly empirical, but its application appears to have given excellent results in practice. The horizontal scales of the river-models used at Vicksburg are usually between 1 : 1,000 and 1 : 600, and the vertical exaggeration of scale about 10 : 1. The reports published from this station are of great interest as showing the agreement between results obtained from the models and those observed in the river.

Comparison of Model Results with those Observed in Nature.

On the hydrodynamical side—the reproduction of currents, water-levels, tides and wave-motions—a suitable model is capable of giving results which may fairly be claimed to be accurate within the limits of experimental investigation.

The question as to how nearly quantitative agreement is to be expected regarding the movement of bed-materials, erosion, and deposition, is not so easy to answer. We know that training works, bridge piers, and other structures affect the set and velocity of the currents to an extent and in a manner which is reproduced with close accuracy in a suitable model. Inasmuch as an increased velocity causes scour and a reduced velocity causes deposition, if the bed-material is moved the movement caused by the change will be in the

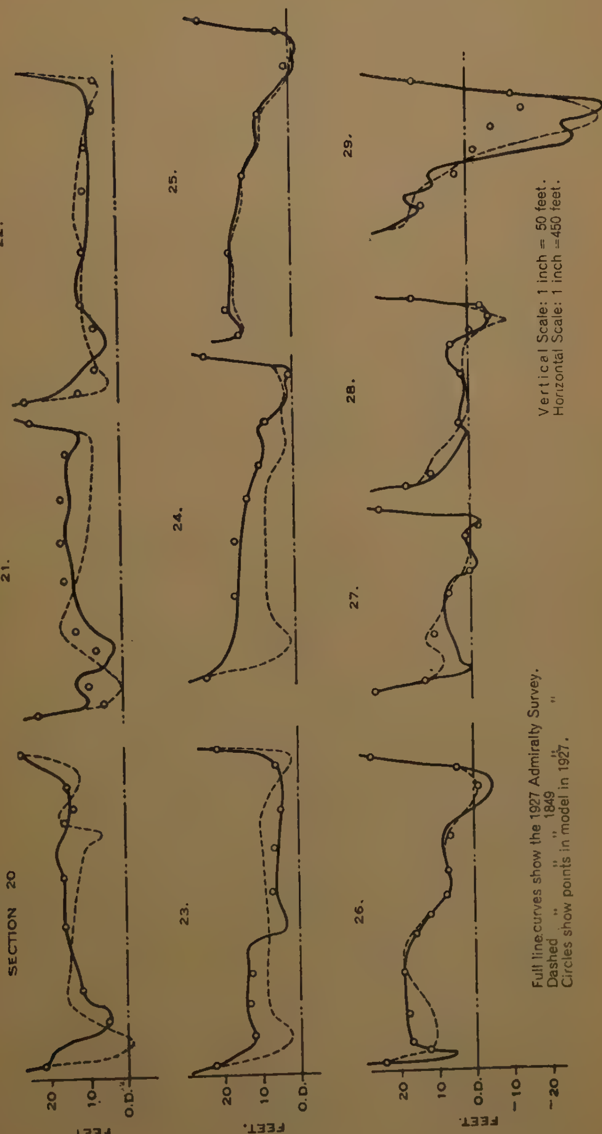
same general direction and of the same kind as in nature, so that it is to be expected that the same kind of configuration will result.

Much depends on how far it is possible in a model to reproduce accurately all the factors involved. Thus in an estuary model, while it is generally possible to reproduce approximately by fans the effect of any prevailing wind, it is impossible to reproduce the effect of gales whose incidence as regards duration and direction is casual, although the general effect of any one gale may be examined. While it is true that in a tidal estuary the scour of the ebb and flood currents is the predominant factor in the maintenance of the deep-water channels, and that over a long period of time where there is no prevailing gale-direction the effect of such occurrences may be expected partially to counteract each other, yet one violent gale may produce changes in an exposed estuary greater than would occur in months or even years of normal ebb and flow. For this reason anything like close detailed agreement between estuary and model over a definite period of years is not probable, except where the estuary is comparatively sheltered and where the physical features are such as to give rise to well-defined and comparatively strong currents.

The final answer to the question can only be given when we have available the results of many model tests and corresponding results under similar conditions in nature, and until then caution must be used in predicting quantitative results.

As yet there are not a large number of such comparisons available, although they are accumulating gradually. Our own experiments at Manchester on the Severn model showed that in such a case it is possible to find a bed-material which will not only reproduce the general behaviour but will also give results which are in reasonable quantitative agreement with nature. The kind of agreement is indicated by the diagrams of *Figs. 2* and *3*, pp. 717-18, which show a number of typical equidistant cross sections in the upper estuary. The broken-line curves show the 1849 survey results, to which the model was moulded. The full-line curves show the 1927 survey, and the circles show points in the model after being operated for a number of tides equivalent to the period between 1849 and 1927.

The upper Severn estuary with its strongly-marked physical characteristics and its rapid currents is an almost ideal subject for model investigation. An estuary with a small tidal range and comparatively feeble currents would not be so suitable, but at the worst such an investigation gives reliable information as to the changes in the tides and in the velocities and directions of the currents, from which valuable deductions as to the probable effect on the bed may be drawn, and which could not be obtained in any other way.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Full line curves show the 1927 Admiralty Survey.
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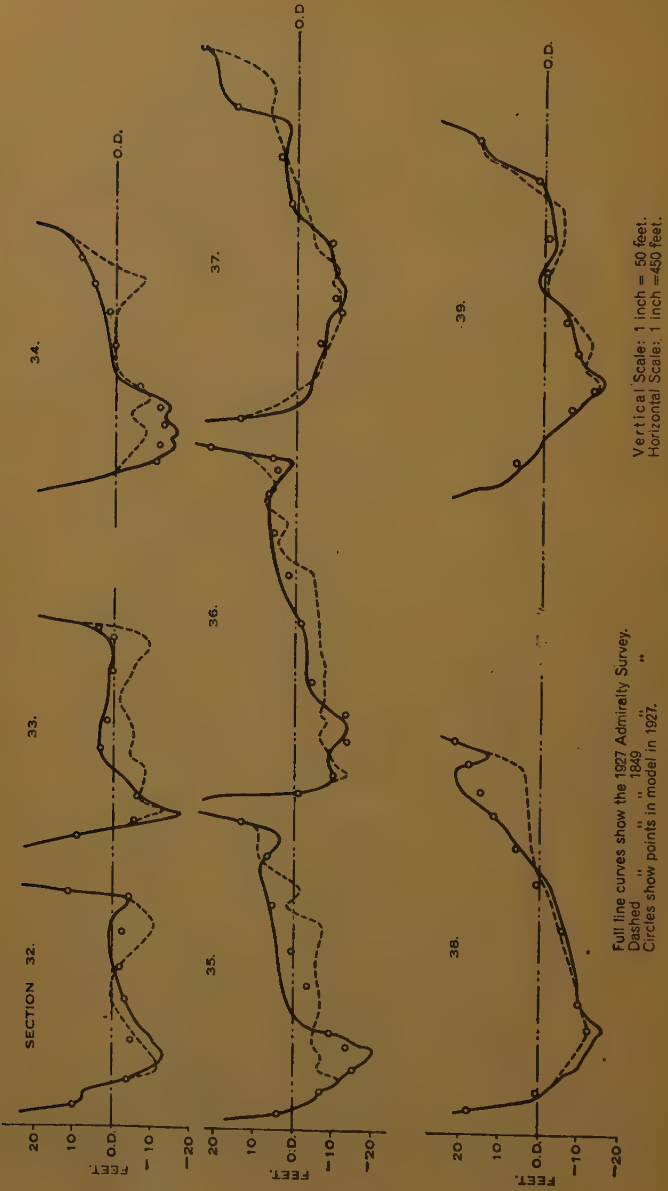
Circles show points in model in 1927.
Vertical Scale: 1 inch = 50 feet.
Horizontal Scale: 1 inch = 450 feet.

Figs. 2.

Figs. 3.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Full line curves show the 1927 Admiralty Survey.
Dashed " " " 1849 "
Circles show points in model in 1927.
Vertical Scale: 1 inch = 50 feet.
Horizontal Scale: 1 inch =450 feet.

Lieutenant H. D. Vogel, in an interesting paper,[1] showed a comparison between the results of tests at the U.S. Waterways Experiment Station, Vicksburg, on models of five different stretches of the river in which spur dykes have been used to train and deepen the navigable channel, and the results observed in the river itself. In every case the agreement is satisfactory, both qualitatively and quantitatively. Excellent agreement has also been obtained during investigations of the effect of cutoffs on the river-levels and on bottom scour, using models of different scales.

The official opinion of the staff of the Experiment Station as to the value of such tests, based on their own experience, is expressed in the following quotation from a recent report: [2] "It has been clearly shown that reliance can be placed on the small-scale hydraulic model to simulate accurately the condition of nature in any particular stretch of river, and that the model can be used as a rational basis for the study of the effects on the river of any improvement works."

A recent comparison between the erosion at the toe of a sluice-dam in nature and in two models to scales of 1/6 and 1/50 has been given by Messrs. Butcher and Atkinson.[3] Here the agreement in each case was very close.

It is perhaps in the type of investigation where a comparison of the relative effect of different schemes is required that the scale model can be used with most confidence. Thus, in a model for investigating the scour at the toe of a projected spillway, even though conditions are such that the actual depth and length of scour are not reproduced with any great accuracy, the relative merits of different designs in reducing scour are likely to be indicated accurately. The same remarks apply to an investigation of the effect of different schemes of training works in a river or estuary.

I think it is not overstating the case to say that in expert hands such models are capable of giving results whose interpretation enables quantitative values to be predetermined with sufficient accuracy for most practical engineering purposes. Much work is now being done in the hydraulic laboratories of the world to put our knowledge of the phenomena of the translation of bed-materials on a sounder basis, and as this knowledge grows it is to be anticipated that it will enable the river model to become more and more nearly an instrument of precision.

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1 "Hydraulic Laboratory Results and Their Verification in Nature." Proc. Am. Soc. C.E., vol. 61 (1935), p. 57.

2 Paper No. 15, U.S. Waterways Experiment Station, Vicksburg, Jan. 1934 (p. 57).

3 *Loc. cit.*

Construction and Operation of Tidal and River Models.

Before constructing a river or tidal model, certain data, to be obtained from field observations, are required.

In a river model it is necessary to obtain :—

(1) Cross sections at frequent intervals over the reach to be investigated.

(2) Estimates of the discharge under normal conditions and in times of drought and flood, with corresponding values of the gauge heights.

(3) Current-velocities at a number of points.

(4) Samples of the bed-material.

For a tidal model it is also necessary to have :—

(5) Tide curves for spring, mean, and neap tides at the seaward end of the estuary, and, if possible, at one or more points in the upper estuary.

(6) Silt samples from the estuary itself taken at a series of depths on the ebb and flood at spring, mean and neap tides, and also silt samples from any river entering the estuary.

(7) Samples of the estuarine water, from which the salinity may be determined at different points and under different tidal conditions.

If charts are available showing changes over a period of years, and especially if they show changes produced by any artificial works in river or estuary, they are invaluable as affording a means of checking the suitability of the bed-material.

For indoor work, the model itself is usually constructed in a wooden casing whose upper edge is carefully planed and levelled so as to represent some definite datum-level. Equidistant cross sections are drawn, preferably on thin cardboard, which is cut out to form male and female templets. Where the charts show sand or mud the lower templets are cut away by an amount equivalent to 15 or 20 feet, so as to allow of a layer of sand of this thickness over the bed. These templets are then fixed vertically in position on the bed of the casing, and the bed and sides of the channel are moulded to them in a weak sand—cement mixture. When this is set, a layer of wet sand of the required grain-size is introduced and is moulded to the correct contours by means of the upper or male templets.

In a model intended to investigate the erosion of the banks, these must be moulded in some erodible material. A mixture of sand and clay may be used for this purpose in such proportions as experiment shows to erode approximately in the same way and at the same rate as in nature.

The tides are produced by the rise and fall of a plunger. This is

driven by an electric motor through an epicyclic gear so designed as to alter the length of successive strokes, so that in each cycle of 28 tides the tidal range varies from springs to neaps and back again to springs.

The tidal curve in an estuary is usually complex, with unequal times of ebb and flow, and to reproduce this with the required degree of accuracy is sometimes a matter of difficulty. The correct tidal range can readily be produced by a plunger of suitable cross-sectional area and stroke, and the required inequality of ebb and flood periods can be obtained by driving the plunger through a pin-and-slot mechanism giving unequal periods to the upward and downward strokes. In order to produce the correct rate of rise and fall at each point of the range, it is often necessary to make final adjustments to the shape of the plunger by trial and error. From the point of view of bed-movement the latter half of the ebb tide and the first half of the flood tide are much the most important parts of the cycle, and every effort should be made to get the rates of rise and fall, and hence the tidal currents, correct at these points. A single plunger will usually enable sufficiently accurate tide curves to be produced, but where there is a very pronounced semi-diurnal variation in the tides it may be necessary to use two plungers, the smaller one being driven at one-half the rate of the larger.

Tidal heights are measured by vertical tide-gauges consisting of ivorine strips graduated to represent feet. Recording tide-gauges or gauges involving the use of floats are not generally satisfactory because of the fact that they usually require to be installed in a tide-way, and the restraint required to prevent lateral motion under the action of the currents introduces frictional forces which may cause inaccuracies. Experience shows that a trained observer can estimate to the nearest 1/100 inch on an open tide-gauge, which on a scale of 1 : 200 represents 2 inches of the actual tide.

When the discharge of any river entering the estuary is known, the corresponding discharge in the model can be calculated in terms of the scale reduction and the correct supply can be fed in through a calibrated orifice in a small supply-tank. If the tank is fitted with a glass gauge-tube this can readily be calibrated to obtain the heads corresponding to flood, mean, or dry-weather flow, and the correct periods of each of these types of flow per year of model operation can be given either by manual adjustment or automatically.

Where the river-water contains silt in suspension the orifice tank should receive its supply of silty water from a large supply-tank which itself contains, say, 12 hours' supply. This tank is kept supplied with water having the correct admixture of silt which is kept in suspension by means of a continuously-operated stirrer.

It usually happens that the rivers entering an estuary are tidal for some considerable distance, and an attempt to incorporate these to the tidal limit as a scale reproduction would mean the scale of the model as a whole being reduced. To avoid this and still maintain the scouring effect of this up-river water on the main channel, each river, beyond the first mile or so of its entry into the estuary, is replaced by a labyrinth having passages of suitable cross section and of the correct total length.

Current-velocities in a model may be measured by floats or by observations on small injections of aniline dye. Where velocities in the river or estuary have been obtained by float observations between given points, it is simple and convenient to make corresponding observations in the model using similar types of float. In a model of a steadily-flowing river a pitot tube or some modification of this can be used to give spot readings of the velocity to correspond to current-meter readings in the river, but in a tidal model, owing to the rapidity with which the velocities are changing, this method is not applicable.

Where alum solution is to be used to reproduce the coagulating effect of the sea-water, this is fed in at the seaward end of the model from a series of supply-tanks, each of which has a float carrying a siphon. The head producing discharge depends only upon the length of the siphon and is independent of the amount of solution in the tank, so that the flow, when once adjusted to the required rate by an outlet-tap at the end of the siphon, remains constant so long as the tank is kept supplied with solution.

On the completion of a run a survey of the model is made by taking spot observations of the depth of the bed below datum level along a series of equidistant cross sections. For this purpose, a straight-edge is placed along the appropriate section resting on the planed edges of the casing whose datum level is known, and depths of the bottom below this level are measured by a gauge graduated to represent feet. From these observations cross sections and contours can afterwards be drawn.

Almost 50 years ago, Osborne Reynolds concluded a paper on tidal models read before the International Congress on Navigation at Frankfort-on-Main with the following words: "I have called attention to these results because this method of experimenting seems to afford a ready means of investigating and determining beforehand the effects of any proposed estuary or harbour works: a means which, after what I have seen, I should feel it madness to neglect before entering upon any costly undertaking." I do not think I could end this Lecture better than by quoting these words and by affirming my agreement with them.

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